Berryessa Creek Element
Coyote and Berryessa Creeks
Flood Control Project
Santa Clara County, California

Appendix B: Engineering and Design

Part I

Hydraulic Analysis of Alternatives

BERRYESSA CREEK PROJECT

APPENDIX B, Part I: Hydraulic Analysis of Alternatives

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CHAPTER 1: INTRODUCTION

This portion of the engineering appendix describes the approach and results of hydraulic modeling efforts for the Berryessa Creek Project under without-project conditions and under project alternative scenarios. Only hydraulic analyses are presented; the supporting hydrology is described in the report *Berryessa Creek Watershed Hydrology Report* by Northwest Hydraulic Consultants (NHC 2003, 2006).

This appendix reports the results of the incremental analysis, preliminary array of alternatives analysis, and the final array of alternatives analysis. The incremental analysis was conducted to determine the viability of various improvements along the study reach. The preliminary array of alternatives was then developed using the information from the incremental analysis and the without-project HEC-RAS model. Finally, the final array of alternatives was narrowed down to include the No Action plan and three project alternatives.

Between when the analysis of the incremental and preliminary array of alternatives were conducted (2006-2009) and the analysis of the final array of alternatives was conducted (2010-2011) the study methodology changed. The changes in methodology take into account recent developments in modeling technology to more accurately reflect the conditions in the study area. The HEC-RAS model was also updated to reflect the latest design of the project located immediately downstream of the study area, the Santa Clara Valley Water District's (SCVWD) Lower Berryessa Creek Project. This report describes both the original hydraulic analysis methodology developed for the GRR and the revised methodology developed for the final array of alternatives. Hydraulic modeling of the Berryessa Creek channel was conducted using the Corps of Engineers HEC-RAS computer program. Due to the length of the study a number of versions of the HEC-RAS programs have been used over the years. Floodplain mapping was conducted using the FLO-2D 2-dimensional modeling software with the approach and results described in *Appendix B: Part II Floodplain Development*.

The GRR study reach extends from just upstream of Old Piedmont Road to just downstream of Calaveras Boulevard. All vertical elevation data referenced in this report, including cross sectional and profile plots, are relative to the NAVD88 datum (2.6' higher than NGVD29). The extreme vertical exaggeration in HEC-RAS profile and section views in this report should be noted (100H:1V or greater in some instances). All cross sections are shown looking downstream, and references to right and left bank are likewise based on downstream views.

Figure 1-1 shows the extent of the study area in relation to the overall watershed area.

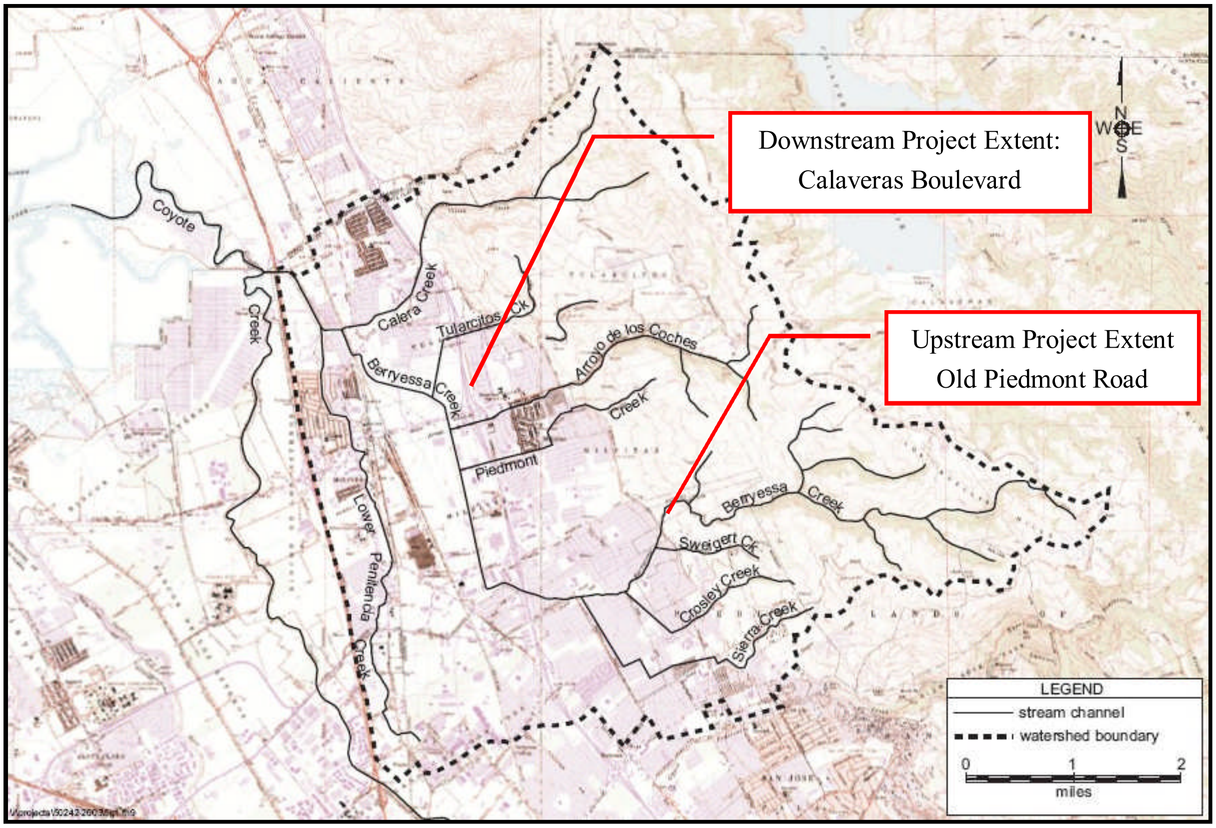

Figure 1-1 Berryessa Creek Study Reach (Source: NHC 2003)

CHAPTER 2: WITHOUT-PROJECT MODEL

The without-project condition was modeled using both the original GRR methodology and the revised GRR methodology in order to ensure that any changes resulting from the change in methodology did not skew the results.

The original GRR without-project conditions hydraulic model was used for the incremental analysis and the development and analysis of the preliminary array of alternatives. The original GRR model was first developed by HDR, Inc. (HDR 2004a) in 2004 with final revisions completed by Tetra Tech in 2009. Changes and updates made to the HDR model are covered in a technical memorandum under separate cover (Tetra Tech 2005a). The preliminary alternative analysis for the study area was completed in 2009 and included the project reaches extending from Old Piedmont Avenue to I-680 for the reach upstream of Interstate 680 (I-680) and I-680 to Calaveras Boulevard for the reach downstream of I-680.

In 2010 and 2011, revisions to the without-project conditions GRR model were carried out. The revisions since 2010 (hereafter called the revised GRR model) are further refinements of the original GRR model. During the analysis of the array of preliminary alternatives it was determined that a federally funded project upstream of I-680 was not justified. Therefore the revised without-project GRR HEC-RAS model was modified to model only the channel reach downstream of the I-680 culvert. The Berryessa Creek channel upstream of the I-680 culvert is now completely modeled by the Upper Berryessa FLO-2D model (see *Appendix B, Part II: Without-Project Floodplain Development*) and the channel reach upstream of I-680 of the HEC-RAS model is not used for the final array of alternatives. The HEC-RAS model was also modified to run in the unsteady mode. Finally, the model reach downstream of the study area (downstream of Calaveras Boulevard) was modified to reflect the Santa Clara Valley Water District's Lower Berryessa Project 60% design.

The following sections describe both the original and revised without-project GRR models. The original without-project GRR modeling is presented to preserve continuity for model results used in the incremental and preliminary alternative array analyses done in the early planning stages of the study that will not be updated for the revised GRR modeling effort.

2.1 Original GRR Model

2.1.1 Model Input

2.1.1.1 Discharge

Watershed delineations, rainfall-runoff relations, and peak flow hydrology were taken from the NHC, Inc. hydrology report (NHC 2003, 2006). Discharges used as input into the hydraulic model are taken from the future conditions values published in the NHC hydrology report (NHC 2003, 2006). Table 2-1 shows the peak discharges used in the without-project model.

Table 2-1 Discharges and flow change locations used as model input

Sta.	Description	Peak Discharge by Percent Chance Exceedance (cfs)								
		50%	20%	10%	4%	2%	1%	0.9%	0.5%	0.2%
362+42	Upstream Extent	240	420	560	830	1090	1430	1540	1820	2130
331+36	Sweigert Creek	260	450	600	890	1180	1530	1640	1960	2300
311+68	Crosley Creek	300	500	700	1000	1340	1740	1875	2220	2600
287+58	Sierra Creek	470	710	830	1260	1630	2140	2250	2660	3140
218+21	Montague Expressway	610	960	1220	1620	2020	2780	2810	3490	4200
174+70	Yosemite Drive	620	990	1170	1770	2200	2910	3000	3580	4290
166+54	Piedmont Creek	830	1350	1600	2450	2990	3800	4010	4520	5230
144+67	Arroyo de los Coches	1090	1730	2050	3040	3740	4700	5150	5490	6480

Source: NHC 2003 and HDR 2004a

These discharges represent fully contained flows. Reductions for existing breakout locations are covered in *Appendix B, Part II: Floodplain Development*. Further details on the underlying assumptions and changes to confluence locations are covered in Tetra Tech (2005a) technical memorandum.

2.1.1.2 Geometry

(a) Cross Sections

The HEC-RAS model developed by HDR includes approximately 200 cross sections within the study reach. Cross sections in the HDR model were generally cut based on a digital terrain model developed from aerial photography with supplemental ground survey conducted by SCVWD in 2004. Adjustments made subsequently by Tetra Tech to without-project conditions cross sections are described in the 2005a technical memorandum. Cross section locations are shown in Figure 2-1.

Figure 2-2 shows the overall channel profile within the study reach. The bed slope ranges from approximately 2% at the upstream end to 0.5% at the downstream end of the study reach. Significant grade breaks are shown in Figure 2-2 below. Localized grade breaks are present at concrete drop structures (just downstream of Old Piedmont Road, just upstream of Morrill Avenue, inside Cropley Avenue Culvert, just upstream of I-680) and at the sedimentation basin downstream of the Piedmont-Cropley Culvert.

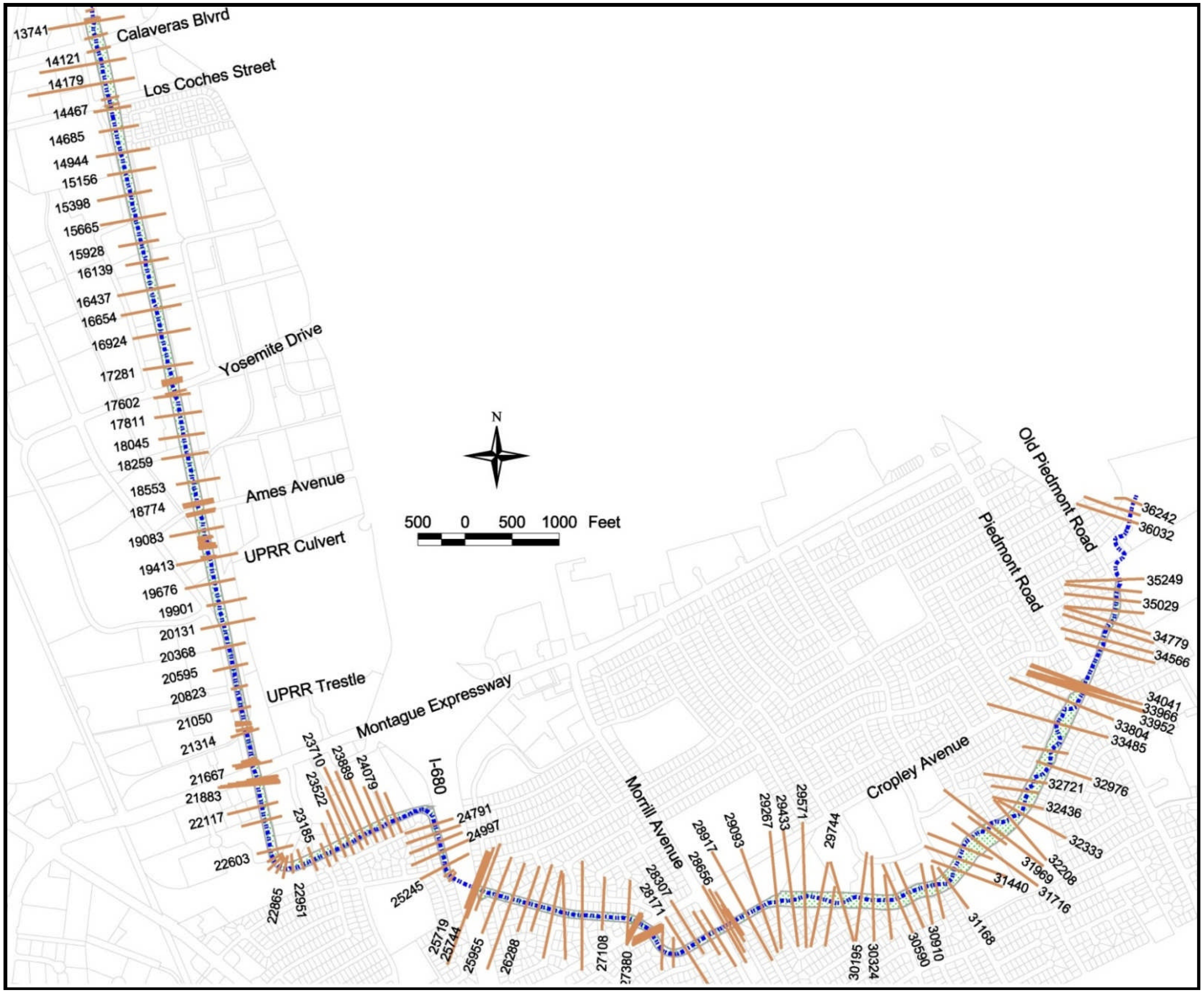

Figure 2-1 HEC-RAS Cross Section Locations (Based on HDR 2004)

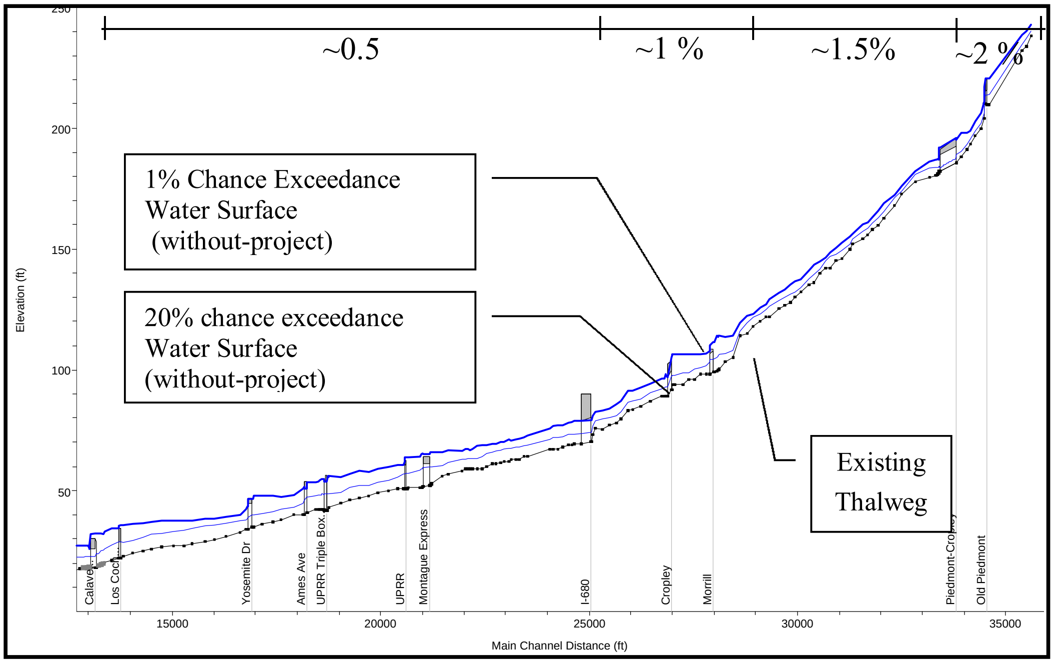

Figure 2-2 Berryessa Creek Profile with Average Bed Slopes

(b) Bridges and Culverts

The without-project conditions geometry file includes twelve structures within the original study reach, as shown in Table 2-2. The four structures upstream of I-680 were subsequently removed from the project area as described below. The without-project conditions model assumes complete maintenance (sediment removal) at bridge and culvert crossings. The effective height of the existing Piedmont-Cropley Culvert, for instance, is modeled as the actual constructed concrete culvert height of 7 feet. Up to 3 feet of sediment deposition has been observed within some of the bridges and culverts, as documented by HDR (2004) and verified through high sediment marks by Tetra Tech during a field visit in October 2004.

Table 2-2 Modeled Bridges and Culverts

HEC-RAS Station	Description	Modeled Type	Approximate Dimensions
351+70	Old Piedmont Road	Bridge	15’ span x 6’ height, irregular opening
342+55	Piedmont-Cropley	Culvert	Single 12’ span x 7’ height box culvert
285+93	Morrill Avenue	Culvert	Double 10’ span x 9’ height box culvert
275+69	Cropley Avenue	Bridge	Double 9.5’ span x 8.5’ height box culvert
255+75	I-680	Bridge	60’ top span x 10’ height, trapezoidal channel
217+38	Montague Expressway	Bridge	Double 12’ span x 9’ height box culvert
212+47	UPRR Trestle	Bridge	40’ top span x 10’ height, 4 sets of piers
193+33	UPRR Culvert	Culvert	Triple 11’ span x 12’ height box culvert
188+43	Ames Avenue	Bridge	75’ top span x 10’ height, trap. channel, single pier
175+18	Yosemite Drive	Bridge	75’ top span x 10’ height, trap. channel, single pier
143+88	Los Coches Street	Bridge	75’ top span x 10’ height, trap. channel, single pier
138+03	Calaveras Boulevard	Bridge	50’ span x 7’ height, 4 continuous piers

As-built bridge plans were obtained for several of the bridges and culverts. A comparison of the plans with observed conditions is presented in Tetra Tech, 2005a, along with changes made to bridges, culverts, and lateral structures for the without-project conditions model. The lateral structures are included in the model to convey overflows; and detailed descriptions and results of overflows are included in *Appendix B, Part II: Without-Project Floodplain Development*. For bridge modeling in the HEC-RAS model, concrete barriers are generally considered part of the bridge deck, while rails are not.

2.1.2 Results

2.1.2.1 Hydraulic Parameters

Table 2-3 shows average hydraulic parameters for the without-project conditions discharges between each set of bridge or culvert crossings. D is the channel hydraulic depth in feet, and V is the average channel velocity in feet per second. These parameters are shown graphically in Figure 2-3 and Figure 2-4.

Figure 2-3 shows that the highest velocities are encountered in the vicinity of the UPRR railroad trestle. Higher localized velocities arise at some of the bridge crossings; however, these higher velocities are offset in the reach-averaged values as flows back up upstream of undersized bridge and culvert entrances. The depths generally increase in the downstream direction as the drainage areas and corresponding peak discharges increase as shown in the Figure 2-3 and Figure 2-4. A comparison of the 50% to 1% chance exceedance event parameters in Figure 2-4 reveals the effect of flows backing up at bridges and culverts. In these areas, the localized 1% chance exceedance velocities decrease and the hydraulic depth increases significantly due to the backwater effect. These figures and tables present results for contained discharges only; that is, the hydraulic parameters presented for any given reach assumes upstream containment measures. Results accounting for breakout flows reducing the channel discharge are presented in *Appendix B, Part II: Without-Project Floodplain Development*.

Table 2-3 Original GRR Model Without-Project Hydraulic Results

Bounding Bridge or Culvert		Percent Chance Exceedance			
		50%		1%	
From	To	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)
Upstream Extent	Old Piedmont Road	6.3	1.8	8.7	4.1
Old Piedmont Rd	Piedmont-Cropley	7.2	1.8	10.7	4.9
Piedmont-Cropley	Morrill Avenue	5.5	2.2	6.6	3.3
Morrill Avenue	Cropley Avenue	5.6	2.6	5.5	6.9
Cropley Avenue	I-680	8.5	2.6	12.5	5.1
I-680	Montague Expressway	5.5	3.1	7.3	5.4
Montague Expressway	UPRR Trestle	7.1	4.1	8.6	7.4
UPRR Trestle	UPRR Culvert	6.9	3.4	9.3	7.1
UPRR Culvert	Ames Avenue	4.6	4.3	7.2	6.6
Ames Avenue	Yosemite Drive	7.0	3.3	6.7	6.4
Yosemite Drive	Los Coches Street	6.0	3.5	5.5	6.4
Los Coches Street	Calaveras Boulevard	6.4	4.7	5.9	8.9
Calaveras Boulevard	Downstream Extent	3.2	4.1	4.2	9.1

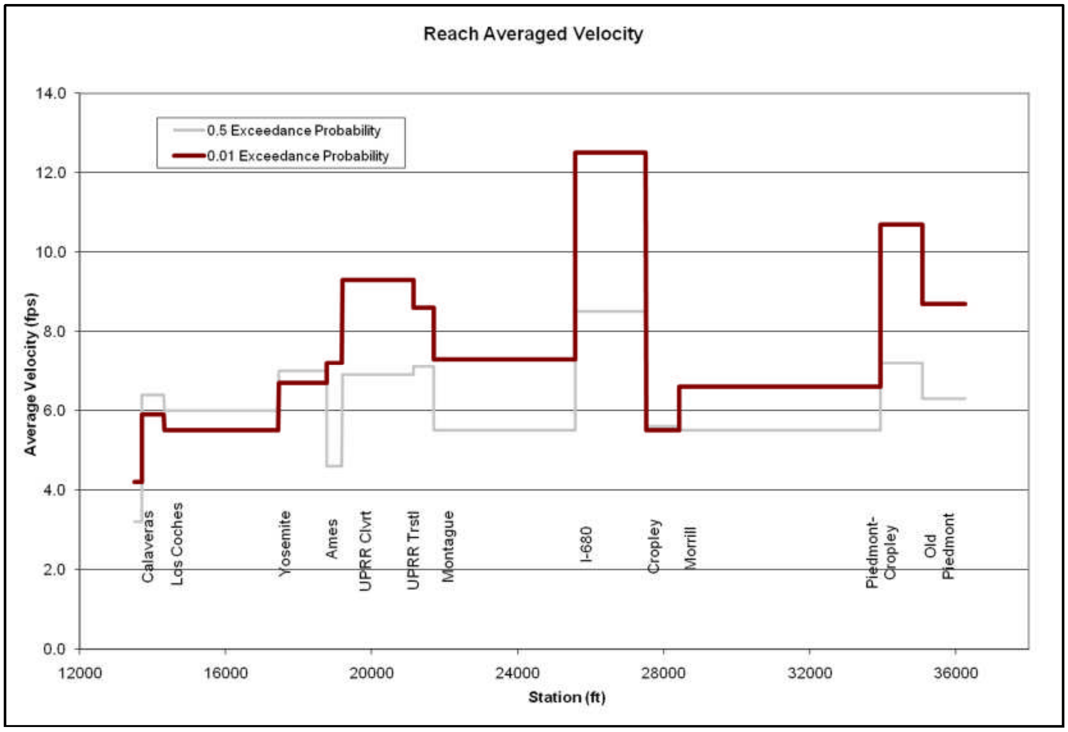

Figure 2-3 Average Channel Velocities between Bridges and Culverts

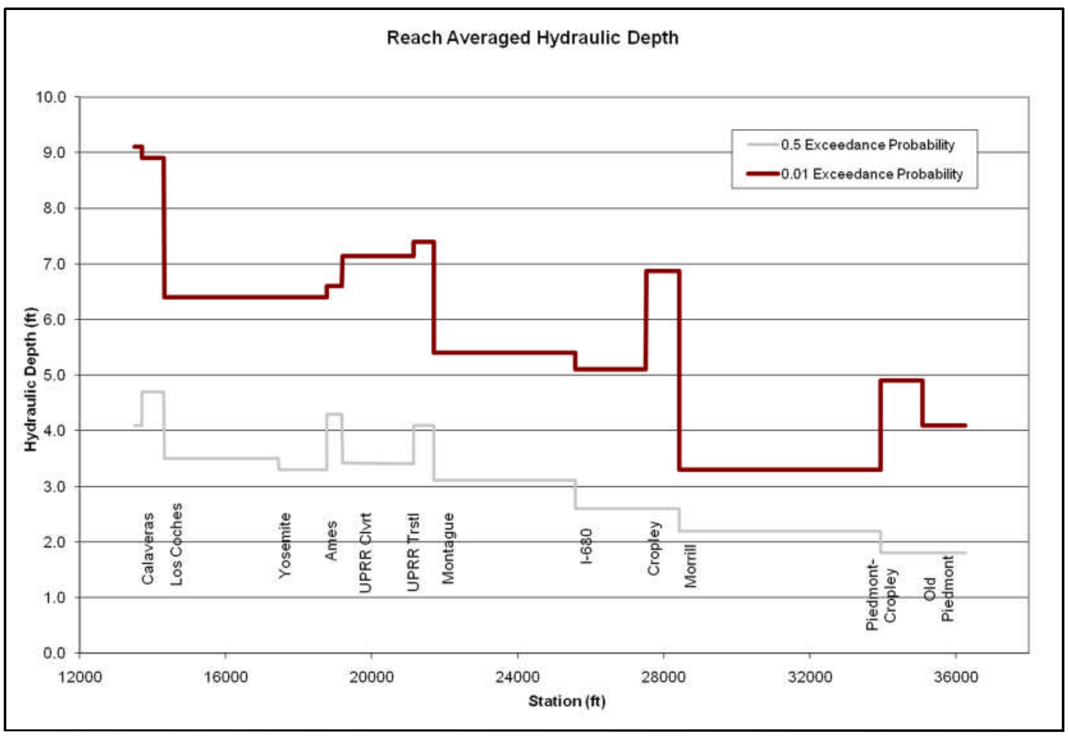

Figure 2-4 Average Hydraulic Depth between Bridges and Culverts

2.2 Revised GRR Model

2.2.1 Model Input

2.2.1.1 Discharge

The conversion of the GRR HEC-RAS Berryessa Creek model from steady to unsteady required the development of hydrographs representing various inflows to the Berryessa Creek Channel. The primary inflow hydrograph to the revised HEC-RAS model is the outflow from the I-680 culvert. The I-680 culvert outflow hydrograph was developed from the output of the Revised Upper Berryessa FLO-2D model (see *Appendix B, Part II: Without-Project Floodplain Development*). The remaining inflow hydrographs to Berryessa Creek consist of subarea runoff and tributary creeks. The inflow hydrographs were taken from the future conditions 2003 HEC-HMS model corresponding to the values published in the NHC hydrology report (NHC 2003). Table 2-4 lists the peak discharges for each inflow hydrograph used in the without-project model, HEC-RAS inflow station and HEC-HMS model nodes used to develop the inflow hydrographs. No changes were made to the hydrology for this study.

The reported discharge hydrographs represent the inflows to the Berryessa Creek channel from I-680 to the confluence with Penitencia Creek. The unsteady HEC-RAS model allows the flows to escape the channel at the existing breakout locations covered in *Appendix B, Part II: Without-Project Floodplain Development*.

Table 2-4 Discharges and flow change locations used as model input

RAS Sta.	HMS Node	Description	Peak Discharge by Percent Chance Exceedance (cfs)							
			50	20	10	4	2	1	0.5	0.2
254+71	-na-	I-680 Outflow from FLO-2D model	490	701	953	1,145	1,403	1,544	1,610	1,771
218+32	B13 RM 3.73	Subarea B12	269	382	461	692	811	928	1,073	1,227
174+48	B15 RM 2.96	Subarea B14	96	149	176	245	275	317	361	414
166+54	B17 RM 2.76	Piedmont Creek	244	387	450	715	821	858	900	900
144+67	B17a RM 2.58	Los Coches Creek	264	429	559	833	868	928	911	951
141+21	B19 RM 2.43	Calaveras Blvd Overflow	0	0	0	0	197	400	400	400
124+03	B21 RM 2.21	Tularcitos Creek	208	332	408	595	652	660	678	685
89+53	B23 RM 1.52	Berryessa Pump	107	150	150	150	150	150	150	150
74+53	B25 RM 1.22	Wrigley-Ford Pump	251	378	432	432	432	432	432	432
59+53	B27 RM 0.94	Calera Creek	180	292	367	521	669	869	1,099	1,261
56+53	B29 RM 0.77	Abbot Pump	583	851	1,041	1,330	1,436	1,568	1,676	1,710
51+53	B31 RM 0.14	Jurgens Pump	127	150	150	150	150	150	150	150
49+74	B 33 RM 0.00	Cal Circle Pump	22	30	34	42	48	56	63	71

Source: NHC 2003

2.2.1.2 Geometry

Two changes were made to the original GRR Berryessa Creek geometry. The first change was to eliminate the model reach and all associated cross sections above cross section 25471. The second was to update the model reach below cross section 13741 to reflect the SCVWD's 60% design for the Lower Berryessa Project. Cross section 25471 represents the outlet of the I-680 culvert and is the upstream end of the revised GRR HEC-RAS model. No changes were made to the channel cross sections, bridges, or culverts between stations 13741 and 25471. The topographic data used in the study area of the HEC-RAS model are derived from 2002 USACE 2' contour interval topography relative to the NAVD 88 datum.

The original GRR HEC-RAS model reach below station 13741 (downstream face of Calaveras Boulevard) was based on the most conservative of the proposed SCVWD Lower Berryessa Project alternatives available during the development of the original GRR model. Since then the SCVWD has designated the Lower Berryessa Project alternative and proceeded to 60% level of design. The SCVWD provided a HEC-RAS model based on the 60% design for the Lower Berryessa Project. The reach downstream of station 13741 in the

SCVWD HEC-RAS model was used to replace the reach downstream of station 13741 in the revised GRR HEC-RAS model. No changes were made to the SCVWD model except for minor changes in hydraulic modeling parameters to facilitate unsteady flow modeling and revising the stationing to match those used in the revised GRR HEC-RAS model.

2.2.2 Results

2.2.2.1 Hydraulic Parameters

Table 2-5 shows average hydraulic parameters for the without-project conditions discharges between each set of bridge or culvert crossings. Depth is the channel hydraulic depth in feet, and Vel is the average channel velocity in feet per second. These parameters are shown graphically in Figure 2-5 and Figure 2-6.

As seen in the Original Model in the previous section, Figure 2-5 shows that the highest velocities are encountered in the trapezoidal reach between the UPPR Trestle and Culvert. Additionally, higher, localized velocities are seen between the Ames Avenue and Yosemite Drive bridges. As with the Original Model, a comparison of the 50% to 1% chance exceedance event parameters in Figure 2-6 show that for the 1% chance exceedance event the bridges and culverts upstream of Yosemite Avenue cause the flows to backup, increasing the flow depths upstream.

Table 2-5 Revised Model Without-Project Hydraulic Results

Bounding Bridge or Culvert		Percent Chance Exceedance			
		50%		1%	
From	To	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)
I-680	Montague Expressway	5.2	3.2	6.1	3.4
Montague Expressway	UPRR Trestle	6.4	4.3	7.0	6.2
UPRR Trestle	UPRR Culvert	6.4	3.4	8.1	5.3
UPRR Culvert	Ames Avenue	4.7	3.7	6.0	5.2
Ames Avenue	Yosemite Drive	6.3	3.2	7.3	3.9
Yosemite Drive	Los Coches Street	5.8	3.6	5.7	3.0
Los Coches Street	Calaveras Boulevard	7.3	3.2	5.3	4.0

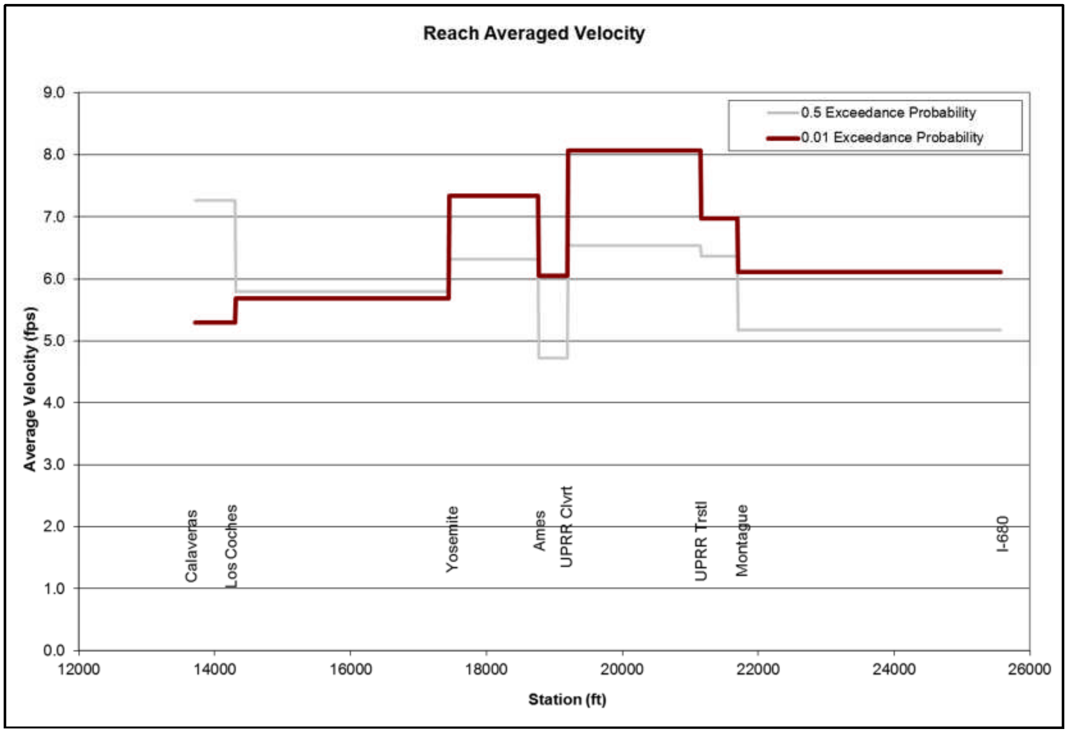

Figure 2-5 Average Channel Velocities between Bridges and Culverts

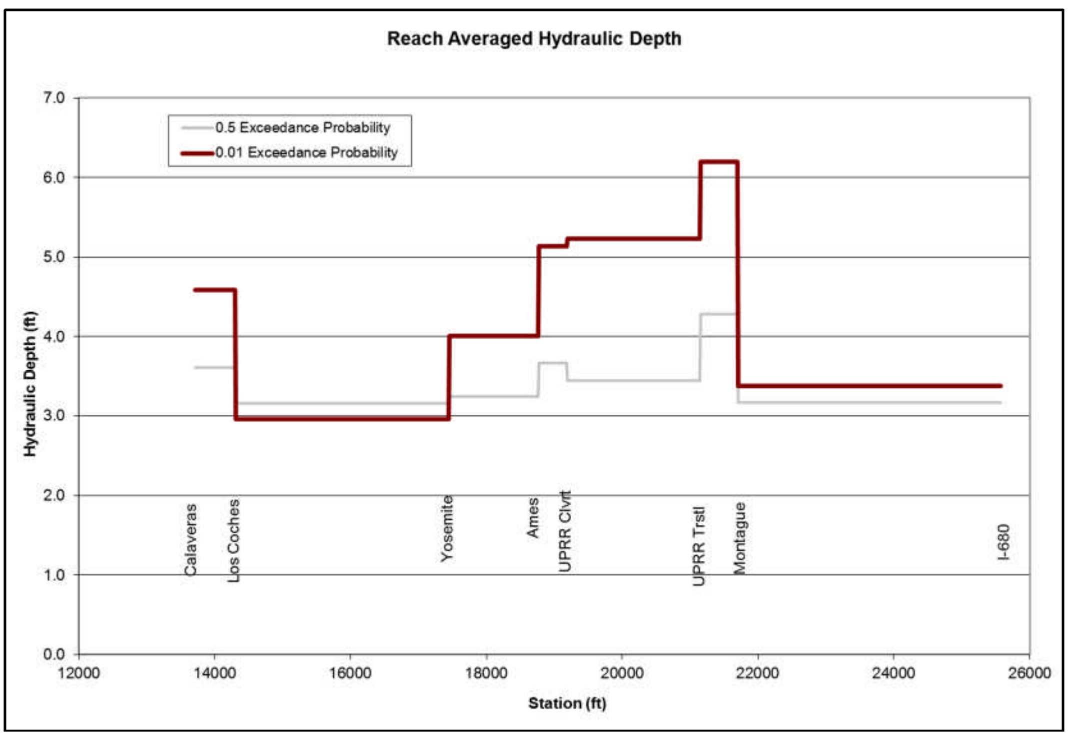

Figure 2-6 Average Hydraulic Depth between Bridges and Culverts

CHAPTER 3: INCREMENTAL ANALYSIS

The incremental analysis was conducted using the original GRR without-project methodology. The original GRR without-project HEC-RAS model contains the 50% chance exceedance event throughout the project reach. Higher discharges begin to break out of the existing channel. In 2006, an incremental analysis was conducted to determine the capacity of each bridge or culvert and intermediate channel reach as well as the action needed to contain each incremental flow from the 50% through the 0.2% chance exceedance events. The original GRR steady flow HEC-RAS model as described in Section 2.1 was used as the basis for the incremental analysis. The incremental analysis is based on the 2003 NHC report and does not account for the updates in the 2006 addendum (NHC 2003, 2006). The incremental analysis was conducted before the final determination was made that there was no justification for federal involvement above I-680. Therefore, the incremental analysis covers the entire study reach from upstream of Old Piedmont Road to Calaveras Boulevard.

3.1 Model Input

3.1.1 Discharge

Adjustments to the model were made cumulatively, so each incremental discharge assumes fully contained conditions (no breakout flows).[1] Overflows are covered separately in *Appendix B, Part II: Floodplain Development*. Discharges for two percent chance exceedance events not published in the NHC hydrology report (NHC 2003) were interpolated between published values to determine intermediate points of overflow. A plot of discharge versus return period was used to ensure that interpolated discharges fell within a smooth curve between computed discharges. Figure 3-1 shows a plot of the curves used to interpolate discharges.

[1] The incremental analysis was conducted before the final determination was made that there was no justification for federal involvement above I-680 and includes analysis of the reach above I-680 not conducted for the final array of alternatives.

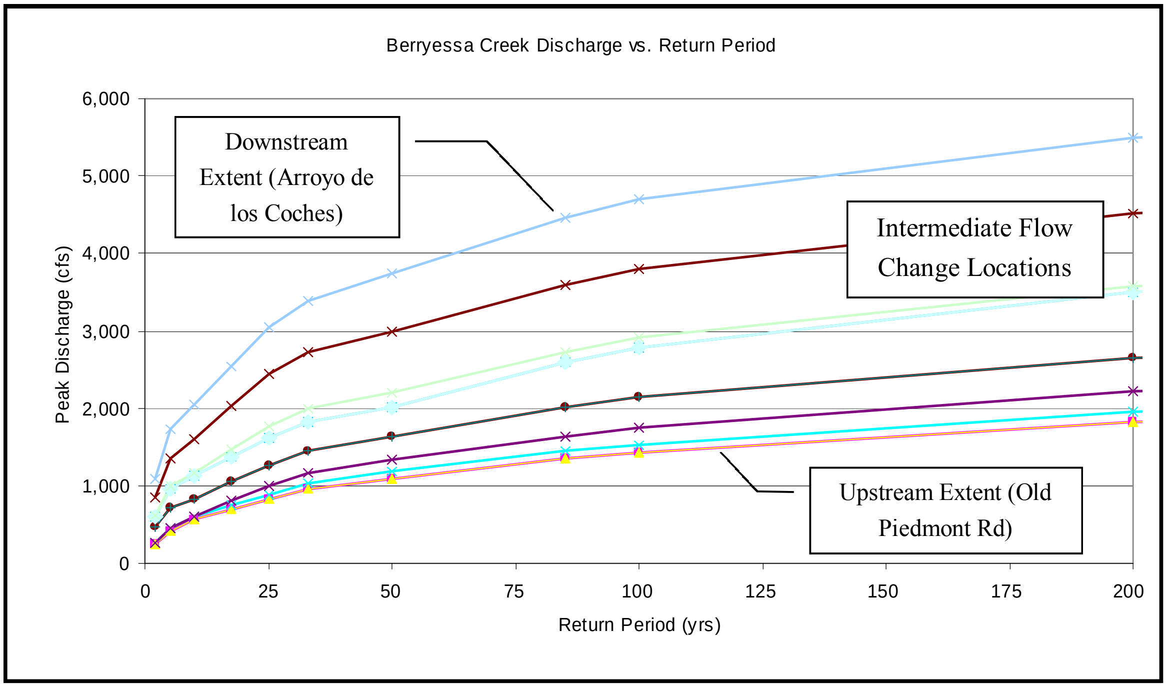

Figure 3-1 Discharge vs. return period for flow change locations

Table 3-1 lists the future conditions discharges published in the NHC hydrology report (NHC 2003) and the interpolated discharges used in the incremental analysis.

Table 3-1 Discharges and flow change locations used for the incremental analysis

HEC-RAS Station	Flow Change Location	Peak Discharge by Percent Chance Exceedance (cfs)									
		50%	20%	10%	5%[1]	4%	3%[1]	2%	1%	0.5%	0.2%
362+42	Upstream	240	420	560	731	830	960	1090	1430	1820	2130
331+36	Sweigert Creek	260	450	600	784	890	1035	1180	1530	1960	2300
311+68	Crosley Creek	300	550	700	1102	1000	1445	1340	1740	2220	2600
286+56	Sierra Creek	470	710	830	1102	1260	1445	1630	2140	2660	3140
218+21	Montague Expressway	610	960	1120	1437	1620	1820	2020	2780	3490	4200
174+70	Yosemite Drive	620	990	1170	2138	1770	2720	2200	2910	3580	4290
166+54	Piedmont Creek	830	1350	1600	2677	2450	3390	2990	3800	4520	5230
144+22	Los Coches Street	1090	1730	2050	3132	3040	3915	3740	4700	5490	6480

Note: 1. Discharges listed in grey columns list discharges interpolated from the reported 2003 values.
Source: NHC 2003

3.1.2 Geometry

(a) Levees

As discharges were incrementally increased in the with-project scenarios, levees were added to cross sections with breakout flows in order to contain the flows. Levees were generally added using the levee function (vertical encroachments) within HEC-RAS, with selected sections modified to ensure that levees with 2:1 side slopes and 12' top widths could be placed within the project footprint without requiring excessive additional height on the levees. In cases where the earthen levees could not be contained within the right of way, vertical concrete floodwalls or additional rights of way are required as described in Chapter 4. Manning's n values for this analysis are described in the following chapter.

(b) Bridges and Culverts

Bridges and culverts were removed from the model individually to quantify the effect on the water surface profile. Individual bridges and culverts with overtopping flows were then resized in conjunction with channel modifications to accommodate each respective incremental discharge. In general, headwall extensions were considered at each bridge or culvert prior to complete replacement; further details on the configuration of the proposed headwall extensions are given in *Appendix B, Part IV: Design and Cost of Alternatives*. The maximum vertical headwall extension was selected as 36" in height. Beyond this threshold, only complete replacement was considered. Replacement spans were attempted in 2' width increments until the discharge passed with no weir flow; pressure flow was allowed to the maximum headwall extension. Capacities listed are for the threshold passing condition without consideration of freeboard requirements.

All bridge and culvert resizing assumes complete maintenance (sediment removal) to the invert as in the without-project models. Bridge design plans from the GDM study were used as the basis for resizing the upstream UPRR trestle. Though the modeled inverts differ from the design plans, the general channel shape from the plans was used in modeling the proposed replacement bridge. Bridge replacement scenarios assume concrete barriers are part of the bridge deck (obstructed), while rails are not.

(c) Channel Excavation

Proposed channel excavation for increased conveyance was generally modeled using the HEC-RAS channel modification function. Channel excavation templates generally follow a smooth slope between existing bridge inverts. Further details on templates for channel modifications are described in Chapter 4.

3.2 Results

This section summarizes the results of modifications to individual bridges and culverts and intermediate channel reaches.[2] The cumulative results of project alternative combinations are presented in Chapter 4. Table 3-2 summarizes the action needed to contain each flow profile by percent chance exceedance. Individual features are presented in order from upstream to downstream. Shading in the table is shown to differentiate channel widening and levees from structural modifications or replacement of bridge or culvert crossings. The corresponding discharges are shown in Table 3-1. Selection of flow profiles for project alternatives was based on the costs of containing each of the incremental flow profiles as described in *Appendix C: Economics*. Table 3-2 shows that earthwork or levee construction begins with the 20% chance exceedance event in a single location and becomes necessary at ten locations for containing the 4% chance exceedance event. Bridge and culvert modifications begin at the 4% chance exceedance event, and full replacement is required at six locations in the 1% chance exceedance event. The results at each feature cannot be interpreted independently, as the size of the channel affects capacities of bridges and culverts, and the size of the bridges and culverts, in turn, affects the capacity of the channel reach.

[2] The incremental analysis was conducted before the final determination was made that there was no justification for federal involvement above I-680 and includes analysis of the reach above I-680 not conducted for the final array of alternatives.

Table 3-2 Action Required to Contain Nearest Percent Chance Exceedance Event

	Percent Chance Exceedance									
Reach/Crossing	**50%**	**20%**	**10%**	**5%**	**4%**	**3%**	**2%**	**1%**	**0.5%**	**0.2%**
Upstream of Old Piedmont Rd	NA	NA	NA	NA	LV	LV	LV	LV	LV	LV
Old Piedmont Road	NA	NA	NA	NA	NA	NA	MC	RC	RC	RC
Old Piedmont Rd to Pied-Crop	NA	NA	NA	NA	NA	BP	BP	BP	BP	BP
Piedmont-Cropley Culvert	NA	NA	GM	GM	GM	MC	MC	RC	RC	RC
Piedmont-Cropley to Messina Dr.	NA	NA	LV	LV	LV	LV	LV	LV	LV	LV
Messina Dr. to Morrill Ave	NA	NA	NA	LV	LV	LV	LV	LV	LV	LV
Morrill Ave Drop&Clvrt+Sierra Cnfl	NA	NA	NA	NA	NA	NA	MC	RC	RC	RC
Morrill Avenue to Cropley Avenue	NA	NA	NA	NA	LV	EX	EX	EX	EX	EL
Cropley Avenue Culvert	NA	NA	NA	NA	NA	MC	MC	RC	RC	RC
Cropley Ave to I-680	NA	NA	NA	NA	NA	NA	NA	NA	LV	LV
I-680	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
I-680 to Montague Expressway	NA	NA	LV	LV	LV	LV	LV	LV	LV	LV
Montague Expressway Culvert	NA	NA	NA	GM	MC	MC	MC	RC	RC	RC
Montague Expy to UPRR Trestle	NA	NA	NA	LV	LV	EL	EL	EL	EL	EL
Railroad Trestle	NA	NA	NA	NA	NA	NA	RC	RC	RC	RC
UPRR Trestle to Culvert	NA	NA	NA	NA	NA	NA	NA	EX	EL	EL
Railroad Culvert	NA	NA	NA	NA	NA	MC	MC	MC	MC	RC
UPRR Culvert to Ames Ave	NA	NA	NA	NA	NA	EX	EX	EX	EX	EL
Ames Avenue Bridge	NA	NA	NA	NA	NA	MC	MC	MC	MC	MC
Ames Ave to Yosemite Dr	NA	NA	NA	LV	LV	EL	EL	EL	EL	EL
Yosemite Drive Bridge	NA	NA	NA	NA	NA	MC	MC	MC	MC	MC
Yosemite Dr to Los Coches St	NA	LV	LV	LV	LV	EL	EL	EL	EL	EL
Los Coches Street Bridge	NA	NA	NA	GM	GM	MC	MC	MC	MC	RC
Los Coches St to Calaveras Blvd	NA	NA	NA	LV	LV	EL	EL	EL	EL	EL
Calaveras Blvd Bridge	NA	NA	NA	GM	MC	MC	MC	MC	RC	RC
Downstream of Calaveras Blvd	NA	NA	NA	NA	NA	EX	EX	EL	EL	EL

Key:

No Action	**NA**
Levee	**LV**
General Maintenance	**GM**
Channel Widening	**EX**
Channel Widening with Levees	**EL**
Bank Protection	**BP**
Modify Crossing	**MC**
Replace Crossing	**RC**

CHAPTER 4: PRELIMINARY ARRAY OF ALTERNATIVES

The preliminary array of alternatives was developed from 2006 to 2009 with the help of the information developed in the incremental analysis. The preliminary array of alternatives analysis was conducted before the final determination was made that there was no justification for federal involvement above I-680. Therefore, the preliminary array of alternatives analysis covers the entire study reach from upstream of Old Piedmont Road to Calaveras Boulevard. The GRR with-project scenarios are built on the original GRR without-project HEC-RAS model and associated assumptions as described in Section 2.1. The preliminary array of alternatives were developed as either a moderate level of protection or FEMA certifiable level of protection to size the project features for the project alternative combinations. The following describes the two levels of protection used in the design of the preliminary analysis:

- *Profile A: Moderate Protection.* Hydraulic structure capacity and levees/top of bank are designed at the water surface level corresponding to the median 0.9% chance exceedance event. The scenario for this level of containment includes channel modifications in addition to modifications and/or complete replacement at bridge and culvert crossings. The modification or retrofitting work includes shoring and transition structures (UPRR Culvert, Ames Avenue Bridge, Yosemite Drive Bridge); headwall extensions with transition structure (Los Coches Street Bridge, Calaveras Boulevard Bridge); and bridge replacement (Old Piedmont Road Bridge, Piedmont-Cropley Culvert, Messina Pedestrian Bridge, Morrill Avenue Culvert, Cropley Avenue Culvert, UPRR Trestle, Montague Expressway Culvert). Modifications within channel reaches include channel widening, bank stabilization, and earthen levee or concrete floodwall construction. Additional details on the individual project features are included in *Appendix B, Part IV: Design and Cost of Alternatives*.

- *Profile B: FEMA Certification Protection.* Risk and uncertainty principles were used in the development of the B alternatives. Hydraulic structure capacity and levees/top of bank are determined according to criteria developed by the U.S. Army Corps of Engineers (Corps) Engineering Circular No. 1110-2-6067 "Certification of Levee Systems for the National Flood Insurance Program," dated September 30, 2008. The Corps HEC-FDA program was used to determine the conditional non-exceedance probability (CNP). The alternatives were broken into reaches, and index points were assigned for each reach. The hydraulic and hydrologic data from the study were input for each index point along with the top of levee elevations to determine the CNP for each reach. Each reach was analyzed according to the above criteria and the top of levee elevations that satisfied the criteria were determined. The resulting elevations from the analyses were then used in the development of the B alternative designs. The scenario for this level of containment involves complete replacement of all bridges and culverts with the exception of the Ames Avenue and Yosemite Drive crossings, which would require shoring/stabilization of existing abutments and construction of transition structures, and the I-680 crossing, which would not be affected. Modifications within channel reaches include excavation and levee/wall construction.

Additional details on the individual project features are included in *Appendix B, Part IV: Design and Cost of Alternatives*.

Further details on the selection of design level of protection are presented in *Appendix C: Economics*. The preliminary alternatives evaluation includes a no action alternative and five project alternatives:

- *Alternative 1 (No Action)*. Without-project condition as described in Section 2.1, assuming routine maintenance.

- *Alternative 2A (Incised Trapezoidal Channel)*. Earthen trapezoidal section with varying bottom width and 2:1 side slopes with a moderate level of containment. Access road intermittently along top of bank or within channel at approximate level of 4% chance exceedance event. Cellular bank stabilization with rip rap toe protection throughout. Levees as required with 2:1 side slopes and 12’ top width.

- *Alternative 2B (Incised Trapezoidal Channel)*. Earthen trapezoidal section with varying bottom width and 2:1 side slopes with a FEMA-certifiable level of containment. Access road intermittently along top of bank or within channel at approximate level of 4% chance exceedance event. The designed level of the maintenance road may vary in order to suit local maintenance needs. Cellular bank stabilization with rip rap toe protection throughout. Levees and floodwalls as required with 2:1 side slopes and 12’ top width. Limited additional right of way.

- *Alternative 3B (Terraced Trapezoidal Channel)*. Earthen section with 10’ bottom width low flow channel, 3:1 side slopes, 3’ deep. FEMA-certifiable level of containment. Two 15’ wide vegetated floodplain terraces. Levees as required with 2:1 side slopes and 12’-18’ top width. Cellular bank stabilization on slopes with rip rap toe protection throughout. Access road along one or both banks, with optional recreational trail. Additional right of way as required.

- *Alternative 4 (Walled Trapezoidal Channel)*. 10’ bottom width earthen low-flow channel with 3:1 side slopes, 3’ deep. FEMA-certifiable level of containment. Two vegetated floodplain benches bounded by vertical concrete floodwalls, 32’ wide on the left bank, and 10’ wide on the right bank. Access road location varies along the top of one or both banks or within channel. Optional recreational trails. Wall extensions as required to contain flows. Limited additional right of way.

- *Alternative 5 (Authorized Plan)*. Levees in the Greenbelt. Concrete trapezoidal channel in downstream of I-680.

All project features upstream of I-680 (including both channel work and bridge and culvert modifications) are consistent among the B alternatives. Bridge and culvert modification and replacement scenarios downstream of I-680 are likewise consistent among the B alternatives; the alternatives differ only in the configuration of the channel reaches between the structures.

Plan views and typical sections showing the overall configuration of each alternative are presented in *Appendix B, Part IV: Design and Cost of Alternatives*.

4.1 Model Input

The following section describes the methodology used in the preliminary array of alternatives analysis.[3]

4.1.1 Flow

Peak discharges for the with-project alternatives were retained from the without-project future conditions hydrology as tabulated above. For comparison purposes, all project conditions models were run both mixed and subcritical, with subcritical results being used to design levee heights and bridge capacities, while mixed run results were used to determine hydraulic parameters for the design of channel and bank stabilization features. All with-project models were checked for convergence in optimized split flow routines as discussed in *Appendix B, Part II: Floodplain Development*.

4.1.2 Geometry

Without-project cross sections were adjusted to reflect the three project alternatives. A description of each typical cross section, including dimensions and side slopes, is presented in *Appendix B, Part IV: Design and Cost of Alternatives*.

Channel excavation was modeled using the HEC-RAS channel modification function. The channel modification routine was run for affected individual sections using composite cut templates with the fill option toggled off (representing excavation only). The option to "daylight once" is also toggled off so that the cut slope is extended along the entire channel. Fill to represent earthen levees was added either as individual cross section points or modeled as vertical levees as applicable. The channel modification routine creates duplicate points in some locations, so the point filter is run with all tolerances set to 0 in order to remove duplicate points. After the routine is run, the new geometry is created. With-project sections were located within the assumed available right of way where possible. Potential discrepancies in the available right of way data are described in Tetra Tech, 2005a.

Manning's roughness coefficients in the Greenbelt reach were retained from the without-project model. A discussion on the selection of n values is included in the HDR report (2004a). Roughness coefficients for project sections downstream of I-680 were assigned using the $n = (n_b + n_1 + n_2 + n_3 + n_4)m$ method as described in EM 1110-2-1601, where n_b is the base value, n_1, n_2, n_3, and n_4 account for surface irregularities, section variation, obstructions, and vegetation, respectively, and m is a coefficient accounting for meandering.

[3] The preliminary array of alternatives analysis was conducted before the final determination was made that there was no justification for federal involvement above I-680 and includes analysis of the reach above I-680 not conducted for the final array of alternatives.

Hardened access roads are assigned a coefficient of 0.02. The designed main channel is assigned a value of 0.03 to reflect a smooth, maintained, earthen channel with grass-lined banks. Vegetated terraces are assigned a coefficient of 0.045. Coefficients for meandering and irregularities are not increased because of the straight nature of this reach and in order to remain consistent with the HDR analysis (2004). Further refinement of Manning's n values is recommended upon selection of vegetation type and density on floodplain benches. Lateral variation in n-values was included in the cross sections to ensure that the water surfaces from composite n values reflect similar water surface elevations. Sensitivity of the water surface elevations to changes in overall cross section roughness was presented in the HDR hydraulics report (2004a). In general, water surfaces are less sensitive to variations in n-values where the water surface profile is controlled by a constricting bridge or culvert. Placement of dense vegetation or lack of maintenance may result in an overall increase in the roughness and require higher levees in some locations. Results of a sediment transport analysis may also require future adjustments to the roughness coefficients in order to simulate meandering, irregularities from channel scour or deposition, and other factors related to the geomorphology of the channel.

Berryessa Creek is earthen channel with the potential for movement of the bed material and changes in the bed form over the course of an event. This change in bed form may impact the roughness of the bed and subsequently the resulting water surface profile. To ensure that the n values used in the HEC-RAS model were reasonable based on bed form type; the Manning's n values used to model the channel were checked against the typical range of Manning's n values for the anticipated bed form type. The anticipated bed form for Berryessa Creek within the project area during high flows is sand dunes based on the anticipated hydraulic conditions and bed sediment type using Figure 5.23 from Sediment Transport Technology by Simon and Senturk (Simon 1992). The typical n value for this type of bed form is 0.02 to 0.04 per Table 4.2 in River Mechanics by Pierre Julien (Julien 2002), Generally, the n values used in the Berryessa Creek HEC-RAS modeling fall within the range of 0.03 to 0.035. This is well within the typical value range for sand dunes. Therefore the n values used are representative of the anticipated bed form type in the Berryessa Creek channel.

4.2 Alternative Development using Risk-Based Project Performance

Project performance for the Berryessa Creek Flood Control Project Post Authorization Study was estimated using the Corps risk-based Monte Carlo simulation program HEC- FDA (Flood Damage Analysis), Version 1.2.4. The HEC-FDA program integrates hydrology, hydraulics, geotechnical and economic relationships to determine damages, flooding risk and project performance.[4] Uncertainty is incorporated for each relationship, and the model samples from a distribution for each observation to estimate damage and flood risk. The Berryessa Creek model includes the following relationships for each economic impact area:

[4] The preliminary array of alternatives analysis was conducted before the final determination was made that there was no justification for federal involvement above I-680 and includes analysis of the reach above I-680 not conducted for the final array of alternatives.

- Probability-Discharge (with uncertainty determined by period of record)
- Stage-Discharge (stage in the channel with estimated error in feet)
- Stage-Damage (not used in this application, starting values added to run program)

The alternatives developed for this study focused on two different levels of protection. The alternative "A" group (Alternative 2A) was developed to pass the 1% chance exceedance event. The "B" category of alternatives, alternatives 2B, 3B, and 4B, were developed to FEMA-certifiable standards as defined in Engineering Circular (EC) 1110-2-6067. The EC lays out the criteria for determining acceptable top of levee elevations in terms of risk-based project performance.

4.2.1 Methodology

(a) Analysis Criteria

Risk and uncertainty principles were used in the development of the 2B, 3B, and 4B alternatives. The goal of the "B" alternatives is to ensure that the alternative designs shall be certifiable for the FEMA National Flood Insurance Program (NFIP). This was done using the criteria presented in the USACE Engineering Circular No. 1110-2-6067 "Certification of Levee Systems for the National Flood Insurance Program" dated September 30, 2008. The criteria for certification of a riverine levee system are as follows:

- The conditional non-exceedance probability (CNP) must be greater than 90% from overtopping of the 1% chance exceedance flood event for all reaches of the levee system.
- If the top of levee elevation if less than three feet above the FEMA base flood elevation, the levee can only be certified if the CNP is greater than 95%.
- The top of levee elevation shall not be less than 2 feet above the FEMA base flood elevation in any event, regardless if the CNP is 95% or greater.

Portions of the Berryessa Creek alternative designs include entrenched channels. EC 1110-2-6067 does not include criteria for entrenched channels. Based on conversations and e-mail correspondence with the Corps (USACE 2008b), for reaches with entrenched channels, the criteria used shall be a minimum bank elevation equal to the 90% CNP at for the FEMA Base Flood Event; with no minimum distance above the base flood for the entrenched channel bank.

In addition to the above criteria for both leveed and entrenched channel reaches, the project evaluation criteria selected is the 0.9% chance exceedance event (1/111 chance) rather than the 1% chance exceedance event. The use of the 0.9% chance exceedance event was selected to provide for robust alternative designs with respect to FEMA certification, against possible future changes in the hydrology or hydraulics. The 0.9% chance exceedance event was selected to ensure that the resulting alternatives would meet the final guidance for entrenched channels when finalized. The guidance was finalized and accepted after this analysis was completed, and this assumption was not carried on for the final array alternatives.

(b) Analysis Method

Risk-based project performance was used to ensure that the alternative designs meet the FEMA certification criteria presented in the previous section. To accomplish this, HEC-FDA version 1.2.4 was used to determine the conditional non-exceedance probability (CNP) for the three "B" alternatives. This section describes the methodologies followed to determine the top of levee elevations and to analyze entrenched channel reaches.

First, each "B" alternative was broken up into reaches and index points were assigned. Then each reach was analyzed as either a leveed or entrenched channel as appropriate. The leveed reaches were analyzed to determine the appropriate top of levee elevation to use for the reach. Entrenched channel reaches were analyzed to determine if the channel would be FEMA certifiable or if levees may be needed. The application for each type of channel is presented below. The results from the analyses were then used in the development of the final design for each of the "B" alternatives.

Leveed Reach

In order to determine the necessary top of levee elevations to satisfy the levee FEMA certification criteria, the following steps were used.

1. The top of levee elevation for each reach was set to the 0.9% chance exceedance event elevation plus 3 feet for each index point and HEC-FDA was run
2. The CNP for the 0.9% chance exceedance event was linearly interpolated from the HEC-RAS CNP output (HEC-FDA output only includes the 10, 4, 2, 1, 0.4, and 0.2 percent chance exceedance events).
3. For reaches with less than a 95% CNP, the top of levee was set at 0.9% chance exceedance stage plus three feet and recorded for that reach. For reaches with greater than 95% CNP the top of levee was revised to 0.9% chance exceedance stage plus two feet and HEC-FDA was run for the revised reaches.
4. The CNP for the 0.9% chance exceedance event was interpolated for the revised reaches.
5. For revised reaches with less than 95% CNP the top of levee was increased by 0.25 ft. For revised reaches with a CNP greater than 95% the top of levee was set to the top of levee elevation and recorded. HEC-FDA was run for the revised reaches.
6. Steps 4 and 5 were repeated until a CNP of greater than 95% was reached and recorded for all revised reaches. If the iterations result in the top elevation of levee returning to the 0.9% chance exceedance stage plus three feet originally used in Step 1, the top of levee elevation is recorded as 0.9% chance exceedance stage plus three feet.
7. The final difference between the 0.9% chance exceedance stage and the top of levee elevation was determined and applied to the all sections of for individual reach.

Entrenched Channel Reach

For the entrenched channel sections the following steps were used.

1. The HEC-FDA top of levee elevation for each reach was set to the 0.9% flood event elevation plus 0.25 feet for each index point and HEC-FDA was run.
2. The CNP for the 0.9% chance exceedance event was interpolated from the HEC-RAS CNP output.
3. For revised reaches with less than 90% CNP the top of levee was increased by 0.25 ft. For revised reaches with a CNP greater than 90% the top of levee was set to the Top of Levee elevation and recorded. HEC-FDA was run for the revised reaches.
4. The CNP for the 0.9% chance exceedance event was interpolated from the HEC-RAS CNP output.
5. Steps 3 and 4 were repeated until a CNP of greater than 90% was reached and recorded for all reaches.
6. The resulting top of levee elevation was compared to the lower of the left and right bank elevation. Reaches in which the lowest top of bank was higher than the final top of levee elevation, the reach passed. Reaches where the top of levee elevation was higher than the lowest top of bank elevation, the bank was considered to have failed, and thus deemed a levee reach and analyzed according to the methodology for levee reaches.

4.2.2 Inputs

In developing a risk-based project performance model a number of different inputs are required. The following inputs were developed for the Berryessa Creek analysis:

- Reaches and index point locations
- Hydrologic
- Hydraulic
- Economic
- Top of Levee Elevation

The following section describes each of the inputs used for the risk based performance in detail.

(a) Reaches and Index Points

Reaches are developed by grouping similar sections of channel into one reach. One representative cross section is chosen for each reach as the index point. This index point is the location where the hydraulic, hydrologic and economic inputs are assigned for that reaches. The Berryessa Creek greenbelt area reaches were determined differently from the balance of the study area.

The Berryessa Creek channel outside of the Greenbelt reach was divided into 9 reaches[5] based on the alternative channel design. The developed channel was divided into reaches based on similar cross sections grouped into one reach. The reach description, upstream bounding section, downstream bounding section and index point are listed in Table 4-1.

Table 4-1 Reach Descriptions for Study Area not including Greenbelt Reach

Reach	Downstream Section	Index Location	Upstream Section	Alternative
Upstream Old Piedmont Rd.	35191	35350	36242	B upstream [1]
Old Piedmont to Piedmont-Cropley	34467	34959	35139	B upstream [1]
Morrill Ave to Cropley Ave	27642	28307	28525	B upstream [1]
Cropley Ave to I-680	25688	26419	27499	B upstream [1]
I-680 to Montague Blvd.	21738	22274	25575	2B, 3B, 4B
Montague to UPRR Trestle	21738	21601	21247	2B, 3B, 4B
UPRR Trestle to UPRR Triple Box	19333	20131	21247	2B, 3B, 4B
UPPR Triple Box to Ames Ave.	18843	19158	19333	2B, 3B, 4B
Ames Ave. to Calaveras Blvd.	13803	16924	18843	2B, 3B, 4B

1. Only one “B” alternative was developed upstream of I-680.

Due to the complexity of the greenbelt area upstream of Interstate 680, reach and index point assignments were done at more frequent intervals. The greenbelt was divided into a number of different reaches based on the cross sections used in the HEC-RAS model. Reaches were developed for each cross section. In locations along the greenbelt with multiple closely spaced cross sections, the sections were grouped together and only one section was analyzed. The reach description, upstream bounding section, downstream bounding section and index point are listed in Table 4-2.

Table 4-2 Reach Descriptions for Greenbelt Reach

Reach	Downstream Section	Index Location	Upstream Section
33966	33904	33966	34041
33773	33756	33773	33804
33485	33480	33485	33490
33378	33370	33378	33380
33166	33136	33166	33207
32976	32889	32976	33136

[5] The hydraulic reaches discussed in this appendix refer to the hydraulic reaches specified in the scope of work to ensure hydraulic performance goals were met. The Economic Appendix discusses the results of the economic analysis on economic reaches developed independently of the hydraulic reaches, based on economic criteria. The reaches referenced in this and the economic appendix are independent and are not meant to correlate between appendices.

Reach	Downstream Section	Index Location	Upstream Section
32877	32753	32877	32889
32721	32659	32721	32753
32645	32631	32645	32659
32580	32575	32580	32585
32436	32430	32436	32440
32333	32330	32333	32339
32208	32200	32208	32210
32097	32090	32097	32100
31969	31960	31969	31970
31905	31900	31905	31910
31716	31710	31716	31720
31571	31570	31571	31572
31440	31322	31440	31559
31168	31160	31168	31170
31078	31070	31078	31080
30965	30910	30965	31026
30808	30800	30808	30810
30720	30720	30720	30731
30590	30580	30590	30600
30478	30470	30478	30480
30324	30304	30324	30327
30195	30190	30195	30200
30043	30040	30043	30050
29983	29980	29983	29990
29873	29870	29873	29880
29744	29740	29744	29750
29571	29570	29571	29580
29433	29430	29433	29440
29199	29093	29199	29267
28917	28910	28917	28920
28758	28749	28758	28770

(b) Hydrologic Inputs

The hydrologic inputs were developed from the Berryessa Creek Watershed Hydrology Report by NHC dated April 2003, amended in October 2006. HEC-FDA allows for the entry of eight standard percent chance exceedance events. The events used were the 50, 20, 10, 4, 2, 1, 0.4, and 0.2 percent chance exceedance events. The data were imported into HEC-FDA from the HEC-RAS using the HEC-RAS water surface profiles import file. The hydrologic data used for each index location is presented in Table 4-1 and Table 4-2.

Confidence limits were applied to the hydrologic data using the guidelines presented in EM 1110-2-1619 "Engineering and Design Risk-based Analysis for Flood Damage Reduction Studies" dated August 1996. An equivalent period of record of 35 years was applied to the hydrologic data for all reaches and was used by the HEC-FDA program to calculate the confidence limits. Table 4-3 lists the hydrologic data used and Table 4-4 lists the hydrologic curve assigned to each reach.

Table 4-3 HEC-FDA Hydrologic Curves Input

Percent Chance Exceedance	Hydrologic Curve					
	1	2	3	4	5	6
50%	240	260	300	470	610	620
20%	420	450	500	710	960	990
10%	560	600	700	830	1220	1170
4%	830	890	1000	1260	1620	1770
2%	1090	1180	1340	1630	2020	2200
1%	1430	1530	1740	2140	2780	2910
0.4%	1820	1960	2220	2660	3490	3580
0.2%	2130	2300	2600	3140	4200	4290

Table 4-4 Reach Hydrologic Curve Assignment

Hydrologic Curve	Reach
1	Upstream of Old Piedmont Rd., Old Piedmont to Piedmont-Cropley, Greenbelt Reaches 33966 to 33166
2	Greenbelt Reaches 32976 to 30590
3	Greenbelt Reaches 30478 to 28917
4	Montague, d/s of Cropley Greenbelt Reaches 28171 to 28758, Morrill Ave to Cropley Ave, Cropley Ave to I-680, Montague to UPRR Trestle
5	Montague to UPRR Trestle , UPRR Trestle to UPRR Triple Box, UPPR Triple Box to Ames Ave.
6	Ames Ave. to Calaveras Blvd.

(c) Hydraulic Inputs

The hydraulic data inputs for each reach were taken from the preliminary HEC-RAS modeling of the alternatives developed for this study. The preliminary HEC-RAS alternative models were run using an "infinite-wall" methodology. The 50, 20, 10, 4, 2, 1, 0.4, and 0.2 percent chance exceedance event stage data were imported into the HEC-FDA model for each index location. An error in the water surface stage was applied to the hydraulic data using the guidelines presented in EM 1110-2-1619 "Engineering and Design Risk-based Analysis for Flood Damage Reduction studies" dated August 1996. The stage error was computed by HEC-FDA using the standard deviation of the error range. The standard deviation was developed using the results from HEC-RAS model runs using high and low Manning's n values for each alternative. The standard deviation was developed from the following equation:

- $S = E_{mean} / 4$ where
- S = standard deviation of error range
- E_{mean} = mean stage difference between high and low Manning's n HEC-RAS runs

The standard deviation of the stage error was applied to the stage-discharge curve increasing linearly up to the stage of the 1% chance exceedance event. The error was set as a constant above the 1% chance exceedance event stage. The hydraulic inputs for the Upper and Lower Berryessa Models are shown in Table 4-5 and Table 4-6.

Table 4-5 Stage for Percent Chance Exceedance Event

Reach	Water Surface Stage for Percent Chance Exceedance Event, ft								Stage Error, ft
	50%	20%	10%	4%	2%	1%	0.4%	0.2%	
Upstream Old Piedmont Rd.	218.6	219.51	219.9	220.52	221.16	221.95	222.79	223.41	0.09
Old Piedmont to Piedmont-Cropley	203.03	203.69	204.11	204.89	205.55	206.05	206.64	207.07	0.18
33966	181.24	182.21	182.82	183.81	184.65	185.59	186.52	187.23	0.17
33773	180.41	181.11	181.54	182.16	182.61	183.05	183.94	183.95	0.28
33485	177.68	178.04	178.28	178.72	179.13	179.60	180.07	180.56	0.27
33378	176.17	176.70	177.09	177.71	178.17	178.66	178.97	179.39	0.38
33166	171.99	172.36	172.61	173.03	173.39	173.81	174.43	174.87	0.09
32976	166.71	167.37	167.83	168.63	169.34	170.09	170.71	171.31	0.13
32877	165.27	165.83	166.22	166.86	167.43	168.21	169.32	169.66	0.11
32721	162.99	163.67	164.11	164.83	165.46	166.14	166.94	167.48	0.16
32645	162.22	162.67	163.00	163.54	164.02	164.54	165.12	165.55	0.13
32580	161.35	161.92	162.27	162.85	163.33	163.86	164.37	164.61	0.38
32436	159.26	159.63	159.92	160.42	160.86	161.34	161.91	162.58	0.20

Reach	Water Surface Stage for Percent Chance Exceedance Event, ft								Stage Error, ft
	50%	20%	10%	4%	2%	1%	0.4%	0.2%	
32333	157.12	157.59	157.89	158.40	158.83	159.27	159.75	160.60	0.22
32208	154.96	155.41	155.71	156.20	156.62	157.13	157.70	158.10	0.27
32097	153.57	154.06	154.38	154.93	155.41	155.96	156.59	156.99	0.37
31969	151.93	152.32	152.57	153.01	153.40	153.82	154.30	154.80	0.12
31905	149.97	150.38	150.66	151.13	151.54	152.00	152.51	152.88	0.19
31716	147.46	148.05	148.43	149.07	149.62	150.21	150.87	151.34	0.38
31571	145.37	145.98	146.37	147.03	147.61	148.23	148.78	149.13	0.36
31440	144.34	144.86	145.21	145.79	146.31	146.93	147.50	147.89	0.34
31168	139.92	140.33	140.60	141.07	141.49	141.95	142.54	143.00	0.13
31078	138.59	139.23	139.66	140.40	141.04	141.75	142.52	143.09	0.26
30965	136.99	137.33	137.56	138.08	138.57	139.11	139.71	140.22	0.24
30808	135.18	135.67	135.99	136.51	136.96	137.44	137.98	138.38	0.35
30720	134.22	134.65	134.94	135.42	135.82	136.25	136.71	137.02	0.22
30590	131.89	132.23	132.46	132.83	133.18	133.55	133.96	134.39	0.14
30478	129.43	129.98	130.46	131.10	131.72	132.39	133.10	133.78	0.40
30324	128.28	128.89	129.38	129.99	130.58	131.22	131.84	132.35	0.41
30195	126.97	127.31	127.60	127.99	128.38	128.79	129.25	129.59	0.12
30043	124.06	124.54	124.93	125.34	125.78	126.28	126.84	127.28	0.34
29983	122.98	123.32	123.61	124.21	124.82	125.41	126.08	126.53	0.35
29873	121.54	122.19	122.72	123.38	124.01	124.53	125.03	125.41	0.47
29744	120.76	121.45	121.95	122.56	123.15	123.62	123.98	124.43	0.45
29571	118.89	119.12	119.45	119.89	120.37	121.12	122.34	123.37	0.36
29433	117.05	117.73	118.34	119.19	120.05	120.92	122.20	123.30	0.18
29199	113.19	113.96	114.62	115.49	116.38	117.34	118.39	119.17	0.36
28917	106.59	107.68	108.52	109.53	110.61	112.41	113.25	114.02	0.35
28758	106.35	107.61	108.15	109.83	111.11	112.93	113.94	114.75	0.19
Morrill Ave to Cropley Ave	218.60	219.51	219.9	220.52	221.16	221.95	222.79	223.41	0.28
Cropley Ave to I-680	203.03	203.69	204.11	204.89	205.55	206.05	206.64	207.07	0.49

Table 4-6 Stage for Percent Chance Exceedance Event

Reach	Water Surface Stage for Percent Chance Exceedance Event, ft								Stage Error, ft
	50%	20	10%	4%	2%	1%	0.4%	0.2%	
Alternative 2B									
I-680 to Montague Blvd.	58.39	58.93	59.30	60.21	61.04	62.61	64.15	66.77	0.49
Montague to UPRR Trestle	55.75	56.57	57.19	58.15	59.10	60.89	62.52	64.10	0.44
UPRR Trestle to UPRR Triple Box	49.44	50.13	50.59	51.28	51.97	53.30	54.58	56.07	0.40
UPPR Triple Box to Ames Ave.	46.27	41.03	47.52	48.19	48.80	49.82	50.78	52.52	0.48
Ames Ave. to Calaveras Blvd.	36.65	37.60	38.02	39.27	40.01	41.37	44.91	47.39	0.63
Alternative 3B									
I-680 to Montague Blvd.	57.92	58.58	59.02	59.91	60.68	62.04	63.30	64.55	0.47
Montague to UPRR Trestle	55.35	56.16	56.70	57.49	58.25	59.64	60.90	62.14	0.30
UPRR Trestle to UPRR Triple Box	49.06	49.80	50.28	50.98	51.66	52.99	54.27	55.79	0.55
UPPR Triple Box to Ames Ave.	46.13	46.95	47.46	48.16	48.77	49.79	50.62	52.38	0.54
Ames Ave. to Calaveras Blvd.	36.38	37.40	37.83	39.08	39.78	40.81	41.77	42.64	0.62
Alternative 4B									
I-680 to Montague Blvd.	58.39	58.93	59.30	60.21	61.04	62.61	64.15	66.77	0.45
Montague to UPRR Trestle	55.75	56.57	57.19	58.15	59.10	60.89	62.52	64.10	0.20
UPRR Trestle to UPRR Triple Box	49.44	50.13	50.59	51.28	51.97	53.30	54.57	56.07	0.61
UPPR Triple Box to Ames Ave.	46.27	47.03	47.52	48.19	48.80	49.82	50.69	52.53	.65
Ames Ave. to Calaveras Blvd.	36.65	37.60	38.02	39.26	40.00	41.10	42.37	43.46	0.73

(d) Economic Inputs

As the name suggests, HEC-FDA is primarily used as a flood damage analysis tool, of which project performance is one aspect. Therefore, economic inputs in the form of stage-damage curves and floodplain structure locations are required. The economic inputs are independent of the project performance results. For analyses performed for this study, one dummy damage curve and one dummy structure were entered into the HEC-FDA model. This economic data consisted of one data point and was used only to allow the calculation of the CNP and did not affect the performance evaluation or represent any particular structure in the floodplain.

(e) Top of Levee Elevations

The top of levee elevations were used as the target for the HEC-FDA program to determine the CNP for each reach of each alternative. A top of levee elevation was entered for all reaches based on the analysis methodology for that reach. The top of levee was based on a height above the FEMA base flood level for all reaches.

The top of levee elevations for the greenbelt area were determined using the HEC-RAS cross section data. The greenbelt index sections were inspected and the left and right top of levee elevations determined. For sections with apparent existing levees, the elevation was taken at the highest point at which the width of the existing ground section was a minimum of 20 feet. For entrenched portions of the channel the top of bank was used. The lower of the left or right bank was taken as the top of levee elevation for the section.

The top of levee elevations for the leveed reaches were then adjusted using the steps described above until the design criteria were met. The top of levee elevations were based on the lowest bank elevation for entrenched channel reaches. The final tops of levee elevations were used as the basis for the final alternative design.

4.2.2.2 Project Performance Results

The risk-based project performance was determined according to the methodologies described above for each reach[6]. Table 4-7 and Table 4-8 list the results for Upper and Lower Berryessa Creek Study areas, respectively. The tables list the reach, type of reach (entrenched or leveed), CNP results for the final successful iteration, height above base flood corresponding to final successful iteration, and required top of bank elevations for leveed reaches.

As seen in Table 4-7, the B alternatives generally meet the design criteria as an entrenched channel in the reach upstream of I-680 (where the project features are identical between the alternatives). This is primarily due to the use of terraces in the greenbelt reach which greatly reduces extent and height of levees required in the Greenbelt reach. The few locations that do require levees correspond to the primary breakout locations, and the majority of the areas showing flooding in the without-project analysis. The height above base flood was applied to any additional cross sections in the specific reach to obtain a similar project performance. In the case of entrenched channels the height above base flood was used to check the top of bank elevation for any additional cross sections in the reach to ensure that they met the minimum acceptable height above base flood for that reach.

[6] The hydraulic reaches discussed in this appendix refer to the hydraulic reaches specified in the scope of work to ensure hydraulic performance goals were met. The Economic Appendix discusses the results of the economic analysis on economic reaches developed independently of the hydraulic reaches, based on economic criteria. The reaches referenced in this and the economic appendix are independent and are not meant to correlate between appendices.

Table 4-7 Risk-Based Project Performance Results Upstream of I-680

Reach	Reach Type	Existing Top of Levee/Bank Elevation, ft	CNP using Channel Criteria	Height above Base Flood	Required Top of Bank using Levee Criteria
Upstream Old Piedmont Rd.	Levee	222.58	0.446	+2.0ft	223.95
Old Piedmont to Piedmont-Cropley	Entrenched	210.58	0.99	+1.25ft	-
33966	Entrenched	189.97	0.99	+1.75 ft	-
33773	Entrenched	185.08	0.99	+1.25 ft	-
33485	Entrenched	182	0.99	+1.25 ft	-
33378	Entrenched	180.78	0.99	+1.25 ft	-
33166	Entrenched	177.39	0.99	+1.25 ft	-
32976	Entrenched	173	0.99	+1.25 ft	-
32877	Entrenched	171.33	0.99	+1.5 ft	-
32721	Entrenched	168.79	0.99	+1.5 ft	-
32645	Entrenched	167.24	0.99	1+.25 ft	-
32580	Entrenched	166	0.99	+1.0 ft	-
32436	Entrenched	162.85	0.96	+1.25 ft	-
32333	Entrenched	161	0.98	+1.5 ft	-
32208	Entrenched	160	0.99	+1.25 ft	-
32097	Entrenched	158.06	0.99	+1.25 ft	-
31969	Entrenched	157.41	0.99	+1.0 ft	-
31905	Entrenched	154.52	0.99	+1.0 ft	-
31716	Entrenched	152.99	0.99	+1.25 ft	-
31571	Entrenched	150.3	0.99	+1.25 ft	-
31440	Levee	147.38	0.478	+2.0 ft	148.93
31168	Entrenched	144.24	0.99	+1.25 ft	-
31078	Entrenched	144.17	0.99	+1.5 ft	-
30965	Entrenched	143.12	0.99	+1.25 ft	-
30808	Entrenched	140.35	0.99	+1.25 ft	-
30720	Entrenched	138.75	0.99	+1.25 ft	-
30590	Entrenched	137.5	0.99	+1.0 ft	-
30478	Entrenched	136	0.99	+1.5 ft	-
30324	Entrenched	133.12	0.9877	+1.25 ft	-
30195	Entrenched	131.22	0.99	+1.0 ft	-
30043	Entrenched	129.58	0.99	+1.25 ft	-

Reach	Reach Type	Existing Top of Levee/Bank Elevation, ft	CNP using Channel Criteria	Height above Base Flood	Required Top of Bank using Levee Criteria
29983	Entrenched	129.57	0.99	+1.25 ft	-
29873	Entrenched	127.97	0.99	+1.25 ft	-
29744	Entrenched	127	0.999	+1.25 ft	-
29571	Entrenched	124.2	0.9958	+2.0 ft	-
29433	Levee	122.2	0.4936	+2.5 ft	123.42
29199	Entrenched	120	0.9959	+2 ft	-
28917	Entrenched	114.42	0.9513	+1.75 ft	-
28758	Levee	112.59	0.1183	+3 ft	115.93
Morrill Ave to Cropley Ave	Entrenched	107.19	0.9992	+1.25 ft	-
Cropley Ave to I-680	Entrenched	90.56	0.9998	+1.5 ft	-

As seen in Table 4-8, all alternatives require the use of levees or floodwalls for certification and purposes downstream of I-680. This is primarily due to large peak flows and limited rights of way through the Lower Berryessa Creek study area. Generally, the reaches passed by meeting the 90% CNP for base flood plus 3 feet requirement. Only for Alternative 3B did the reach above Montague Blvd. exceed a CNP of 95% to allow the use of a base flood plus 2.5 ft for certification. In addition, a short reach of Alternative 4B between Montague and the UPRR Trestle required a base flood plus 3.1ft for certification. The height above the base flood as listed in Table 4-8 was applied to the remainder of the cross sections in the reach to ensure that they meet the minimum acceptable height above base flood for that reach.

Table 4-8 Risk-based Project Performance Results Downstream of I-680

Reach	Reach Type	Base Flood Water Surface, ft	Top of Levee Elevation, ft	Height above Base Flood	CNP for Final Iteration
Alternative 2B					
I-680 to Montague Blvd.	Levee	62.6	65.61	+3ft	0.9123
Montague to UPRR Trestle	Levee	60.9	63.9	+3ft	0.9077
UPRR Trestle to UPRR Triple Box	Levee	53.3	56.3	+3ft	.9604
UPPR Triple Box to Ames Ave.	Levee	49.8	52.82	+3ft	.9615
Ames Ave. to Calaveras Blvd.	Levee	41.4	44.34	+3ft	.9675
Alternative 3B					
I-680 to Montague Blvd.	Levee	62.04	64.54	+2.5ft	.9617
Montague to UPRR Trestle	Levee	59.64	62.64	+3ft	.9850
UPRR Trestle to UPRR Triple Box	Levee	52.99	55.99	+3ft	.9451
UPPR Triple Box to Ames Ave.	Levee	49.79	52.79	+3ft	.9672
Ames Ave. to Calaveras Blvd.	Levee	47.08	50.08	+3ft	.9983
Alternative 4B					
I-680 to Montague Blvd.	Levee	65.61	62.61	+3ft	.9127
Montague to UPRR Trestle	Levee	60.89	63.99	+3.1ft	.9011
UPRR Trestle to UPRR Triple Box	Levee	53.3	56.3	+3ft	.9450
UPPR Triple Box to Ames Ave.	Levee	49.82	52.82	+3ft	.9459
Ames Ave. to Calaveras Blvd.	Levee	41.1	44.1	+3ft	.9550

4.3 Results

This section summarizes the hydraulic characteristics of project conditions alternatives. Further details on cross sections, quantities and costs are included in *Appendix B, Part IV: Design and Cost of Alternatives*. All project features were modeled individually to determine the associated hydraulic effects prior to combining the features into composite with-project alternative models.[7] Summary results of hydraulic parameters are presented in Table 4-9 with additional details in Table 4-10.

[7] The incremental analysis was conducted before the final determination was made that there was no justification for federal involvement above I-680 and includes analysis of the reach above I-680 not conducted for the final array of alternatives.

Table 4-9 With-Project Hydraulic Results Summary

Bounding Bridge or Culvert		1% Percent Chance Exceedance											
From	To	2A		2B		3A		3B		4A		4B	
		D (ft)	V (fps)	D (ft)	V (fps)	D (ft)	V (fps)	D (ft)	V (fps)	D (ft)	V (fps)	D (ft)	V (fps)
Upstream Extent	Old Piedmont Rd	5.4	2.4	7.0	3.9	7.7	6.2	8.4	3.4	7.8	4.8	8.4	3.4
Old Piedmont Rd	Piedmont-Cropley	5.7	1.9	7.2	3.1	8.1	4.9	9.0	5.0	8.6	4.8	9.0	5.0
Piedmont-Cropley	Morrill Avenue	4.9	1.8	5.8	2.3	6.3	3.0	6.4	3.1	7.2	3.6	6.4	3.1
Morrill Avenue	Cropley Avenue	5.2	2.4	5.8	3.4	4.6	5.8	4.7	6.9	5.6	7.6	4.7	6.9
Cropley Avenue	I-680	7.6	2.4	9.0	3.2	11	4.7	11	4.7	11	5.4	11	4.7
I-680	Montague Expy	5.2	3.1	5.8	3.9	6.5	4.9	6.8	5.3	7.2	6.2	6.8	5.3
Montague Expy	UPRR Trestle	6.1	4.3	7.2	5.5	7.4	7.2	7.6	7.3	8.0	8.6	7.6	7.3
UPRR Trestle	UPRR Culvert	6.4	3.4	7.4	4.6	8.5	6.1	8.6	7.1	7.4	7.9	8.6	7.1
UPRR Culvert	Ames Avenue	4.2	3.9	5.1	5.0	6.1	5.9	6.2	6.3	6.3	7.5	6.2	6.3
Ames Avenue	Yosemite Drive	6.3	3.2	7.2	4.4	6.0	5.3	5.8	6.3	6.3	7.6	5.8	6.3
Yosemite Drive	Los Coches Street	5.8	3.5	5.7	4.1	5.5	5.7	5.4	6.3	5.8	7.6	5.4	6.3
Los Coches St	Calaveras Blvd	6.8	3.8	6.8	5.4	6.6	6.5	6.3	7.1	6.8	8.2	6.3	7.1
Calaveras Blvd	Downstream Extent	5.1	3.0	6.4	4.2	7.7	5.7	8.1	6.3	8.3	7.8	8.1	6.3

These results are for fully contained flows. Comparison to existing conditions is therefore hypothetical only; the computed without-project water surface elevation at any point assumes full containment at each upstream section, and flows are restricted to the extent of each cross section in the event of breakout. Results accounting for breakout flows are presented in *Appendix B, Part II: Floodplain Development*, and *Appendix B, Part III: Geomorphology*.

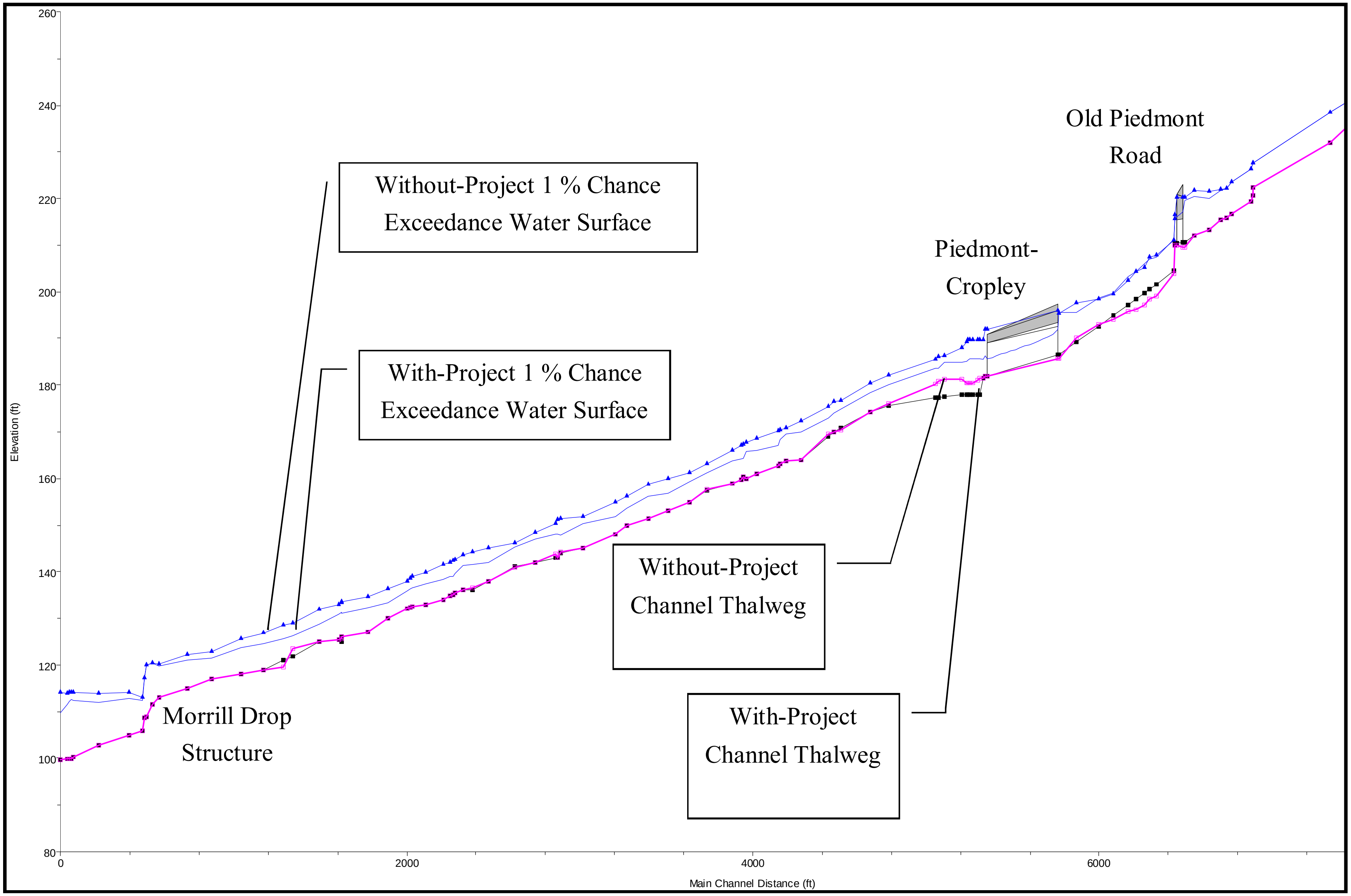

Figure 4-1 Water Surface Profile U/S of I-680, Without-Project vs. Alt 2A

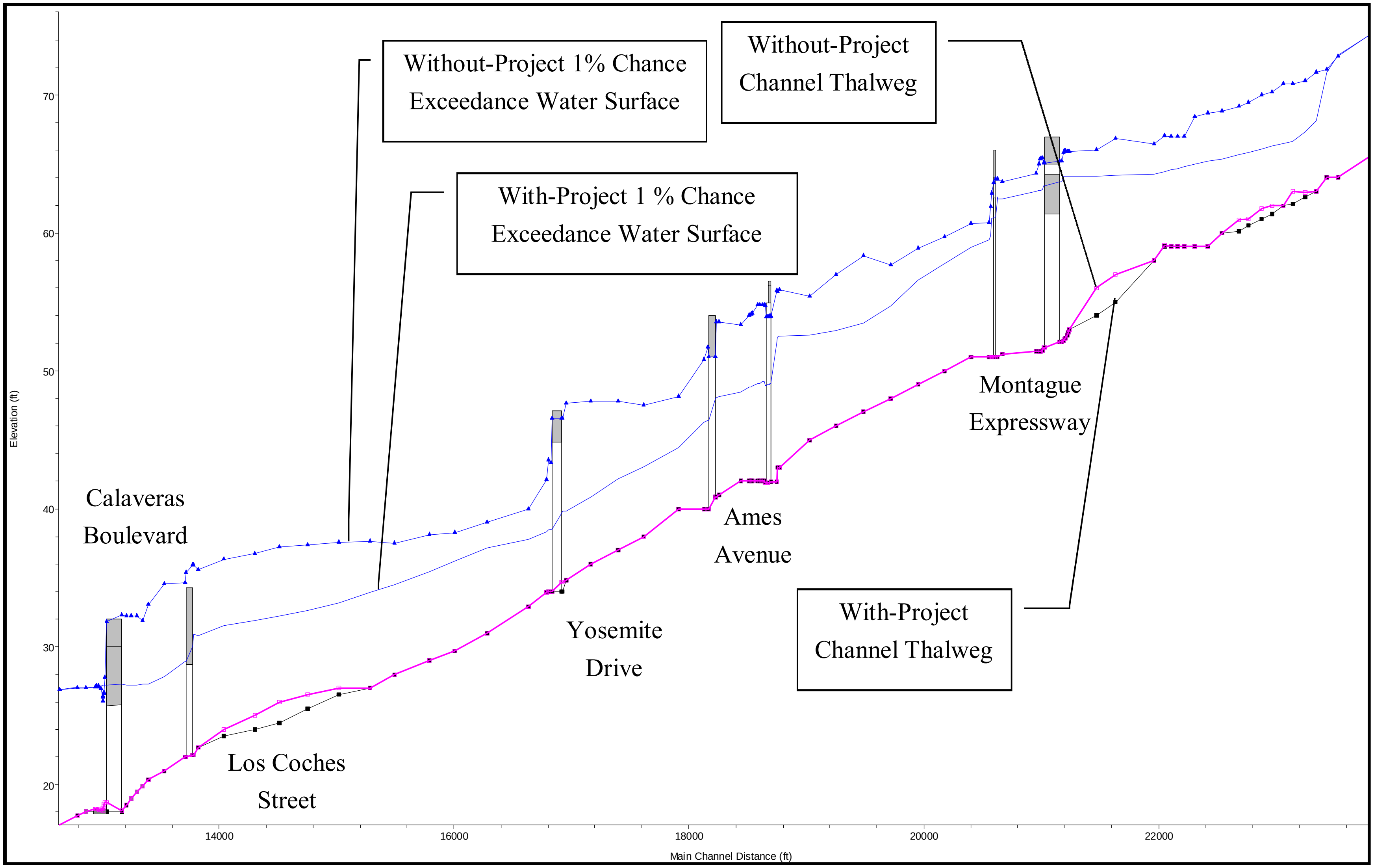

Figure 4-2 Water Surface Profile D/S of I-680, Without-Project vs. Alt. 2A

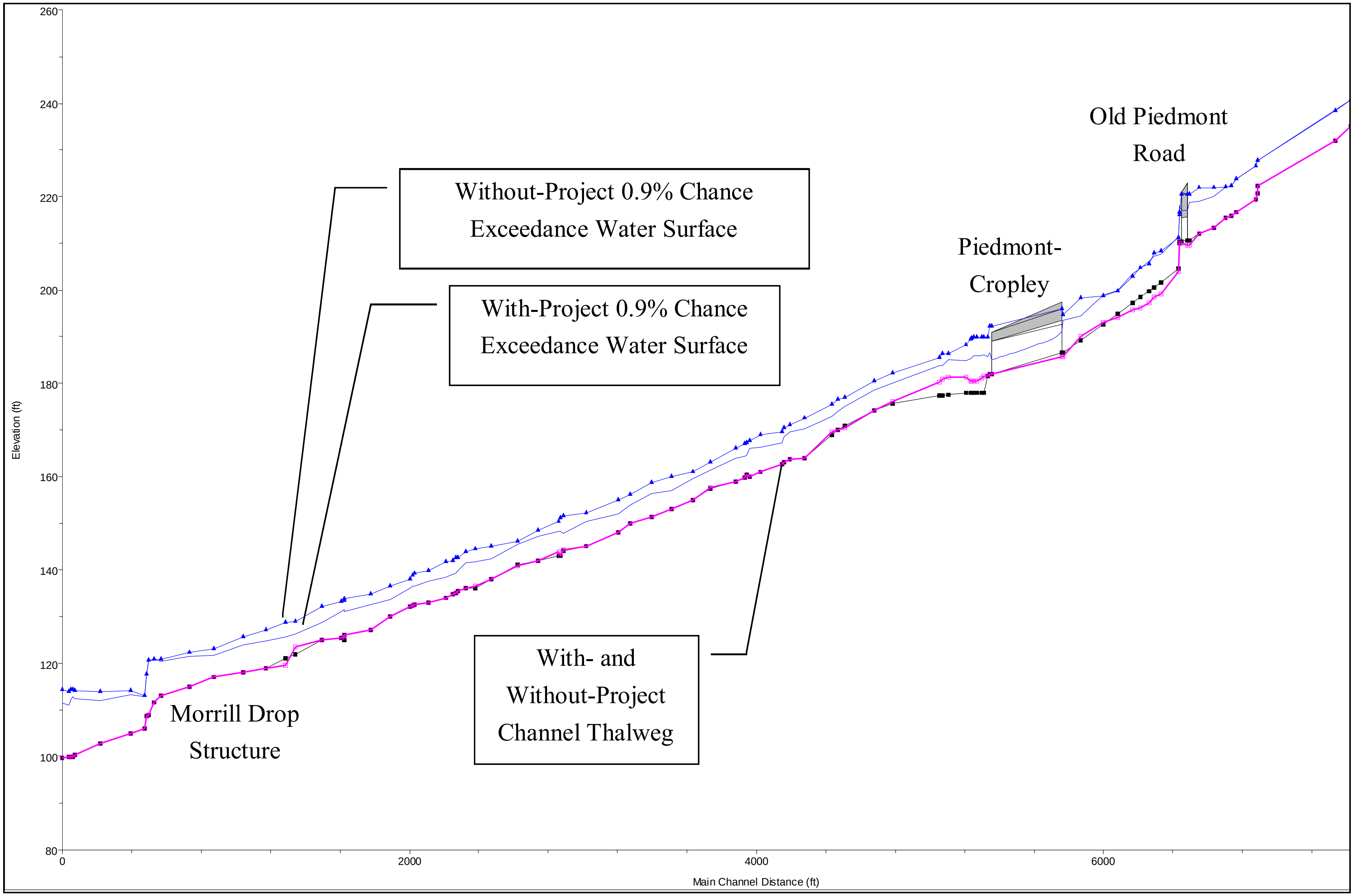

Figure 4-3 Water Surface Profile U/S of I-680, Without-Project vs. B Alternatives

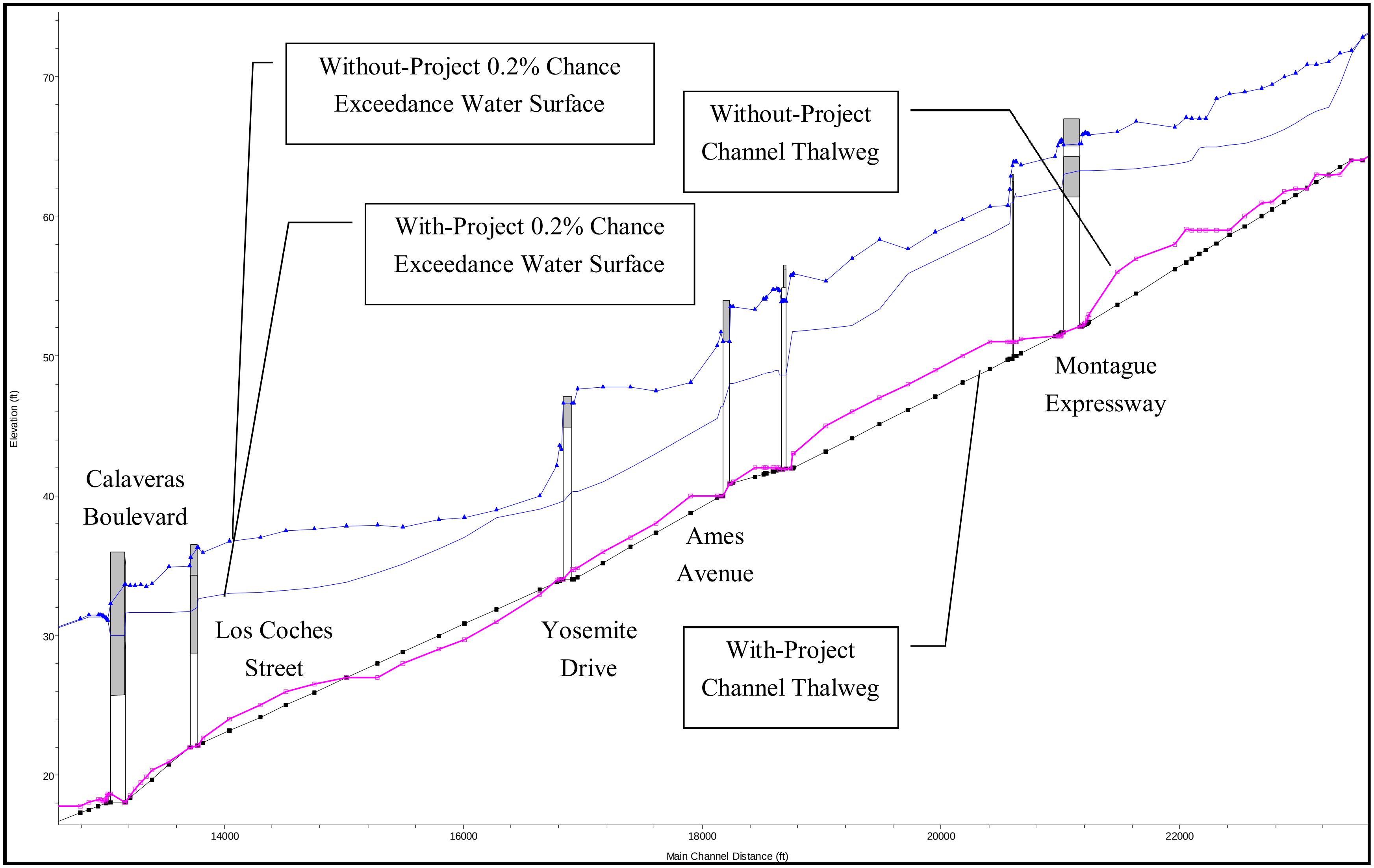

Figure 4-4 Water Surface Profile D/S of I-680, Without-Project vs. Alt. 2B

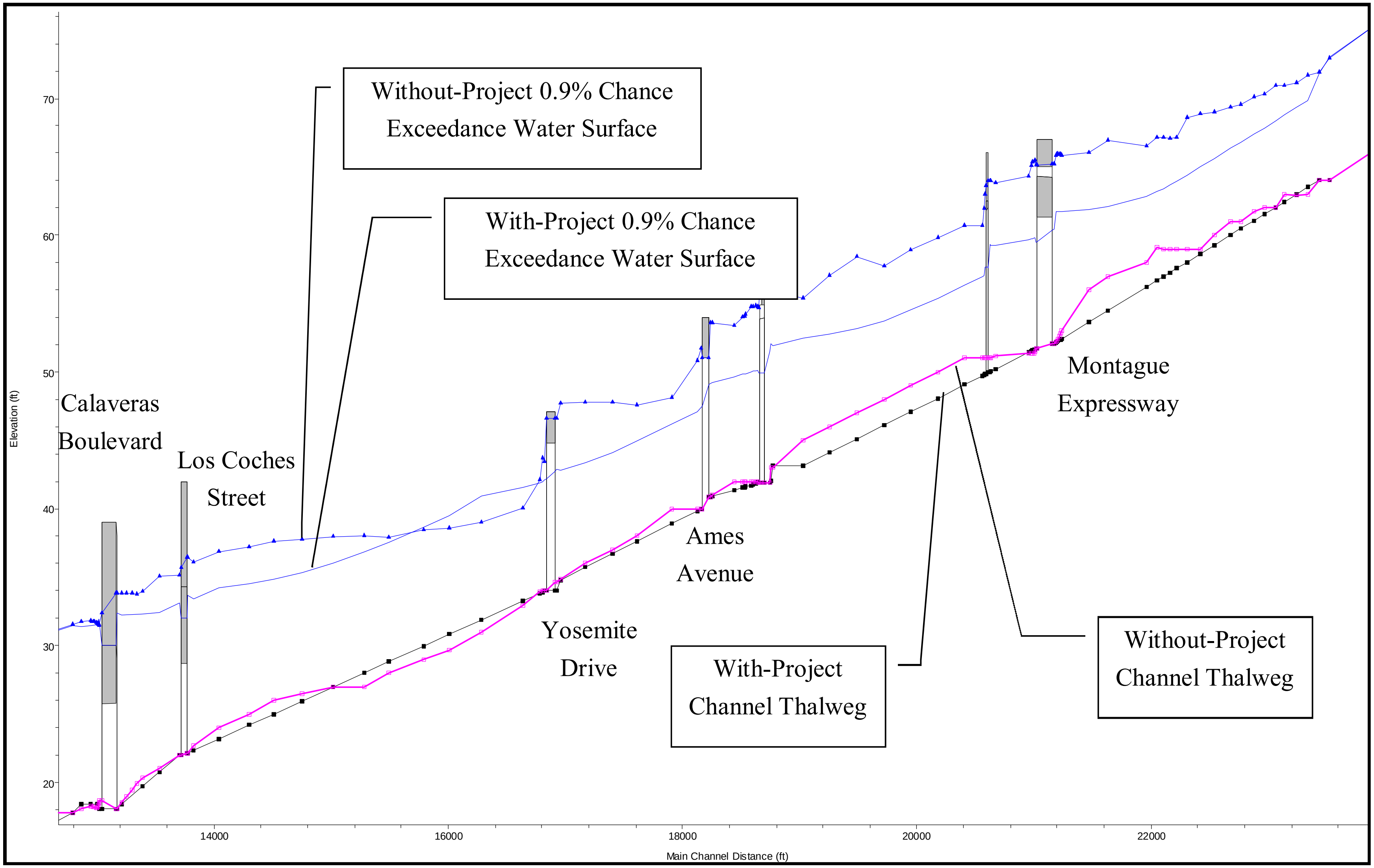

Figure 4-5 Water Surface Profile D/S of I-680, Without-Project vs. Alt. 3B

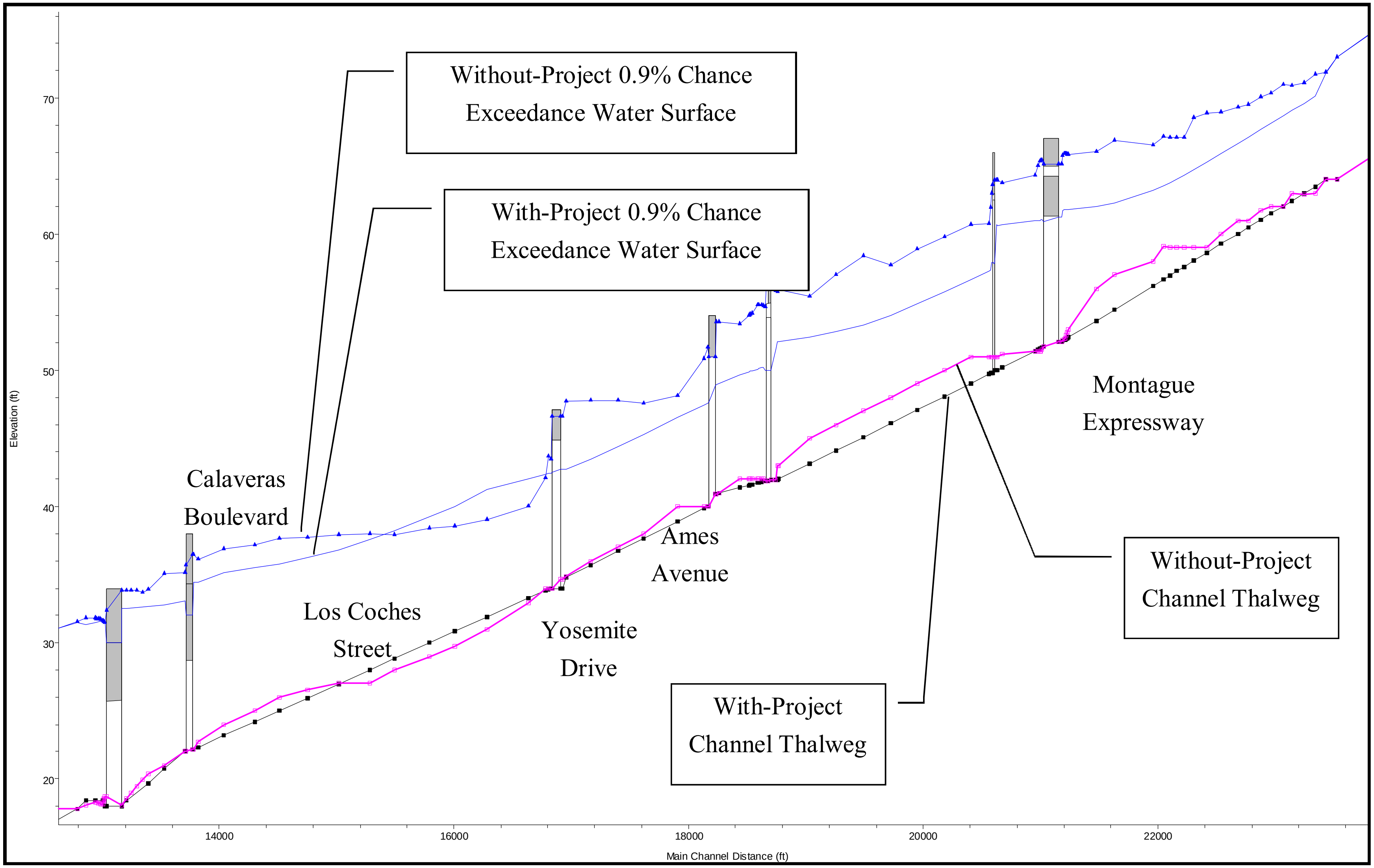

Figure 4-6 Water Surface Profile D/S of I-680, Without-Project vs. Alt. 4B

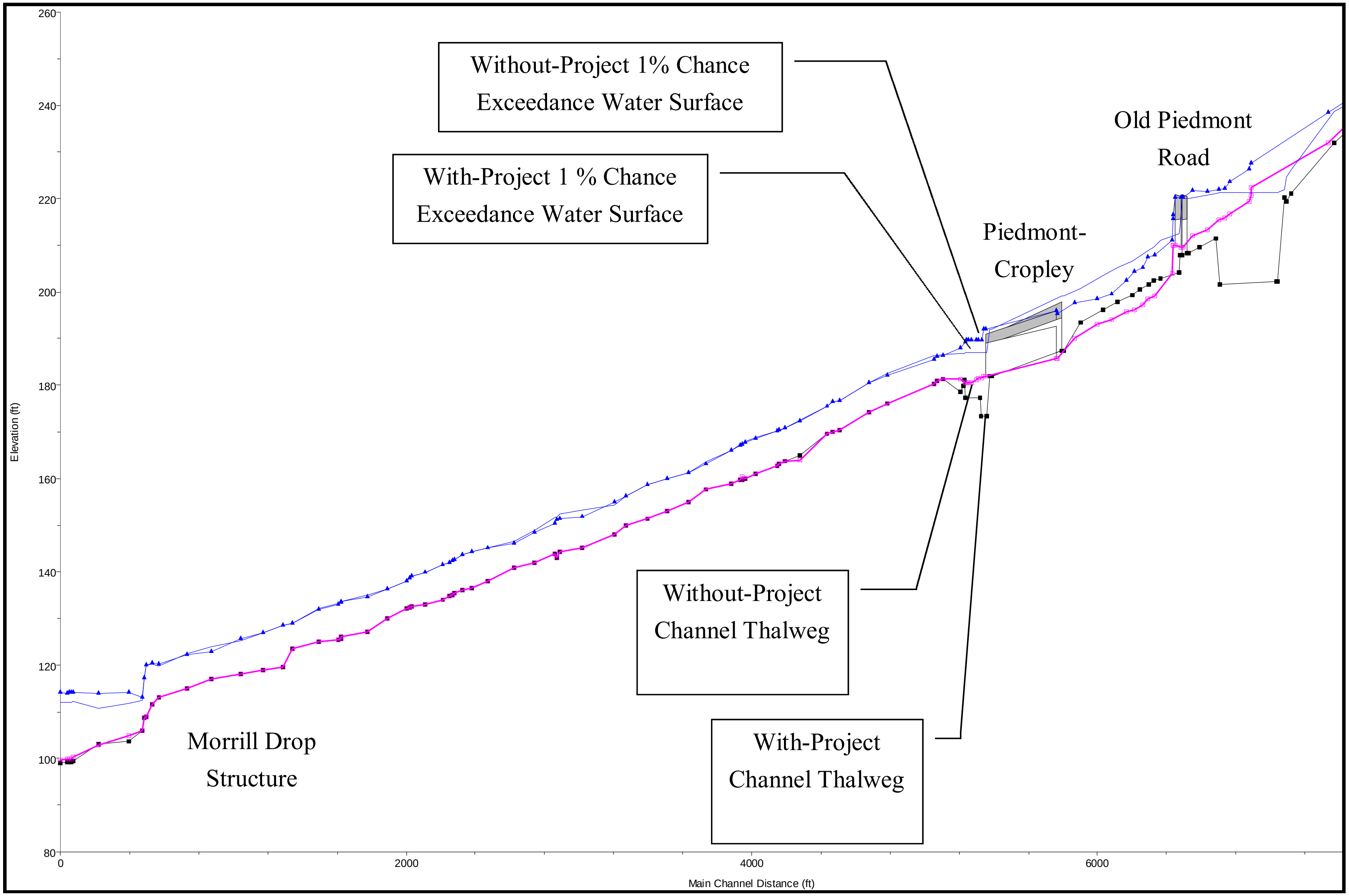

Figure 4-7 Water Surface Profile U/S of I-680, Without-Project vs. Alt. 5

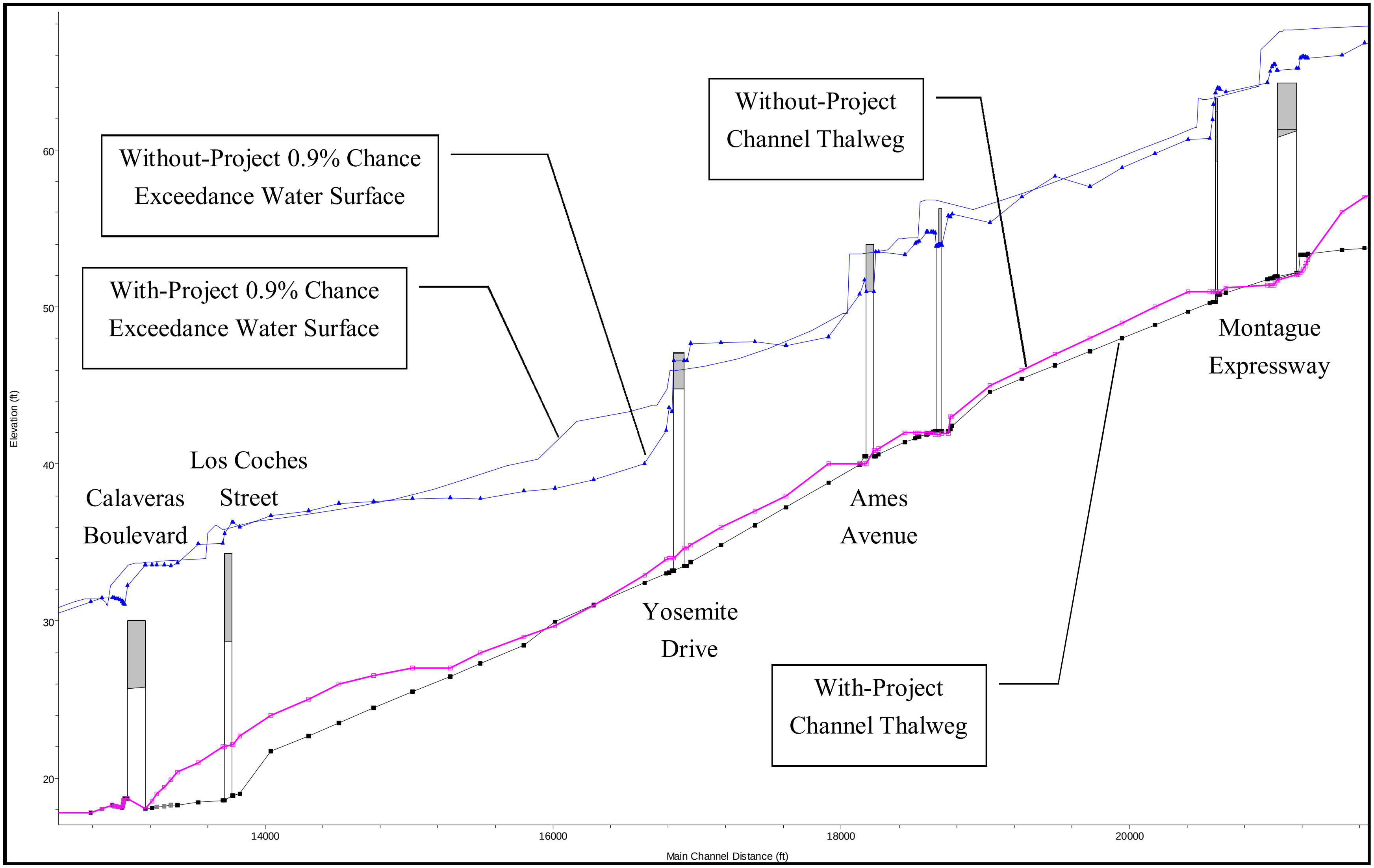

Figure 4-8 Water Surface Profile D/S of I-680, Without-Project vs. Alt. 5

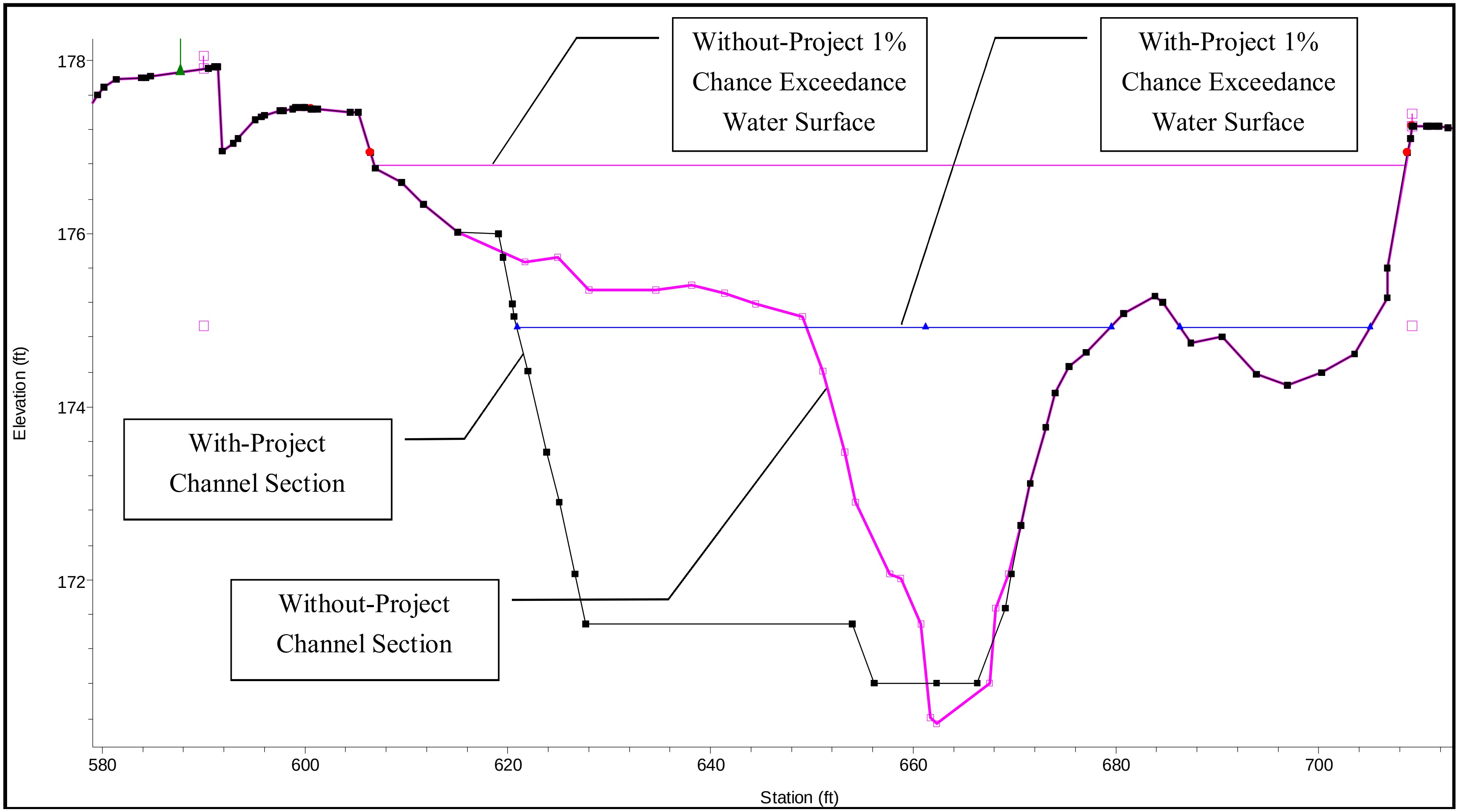

Figure 4-9 Typical U/S section, with- and without-project conditions for Alt 2A

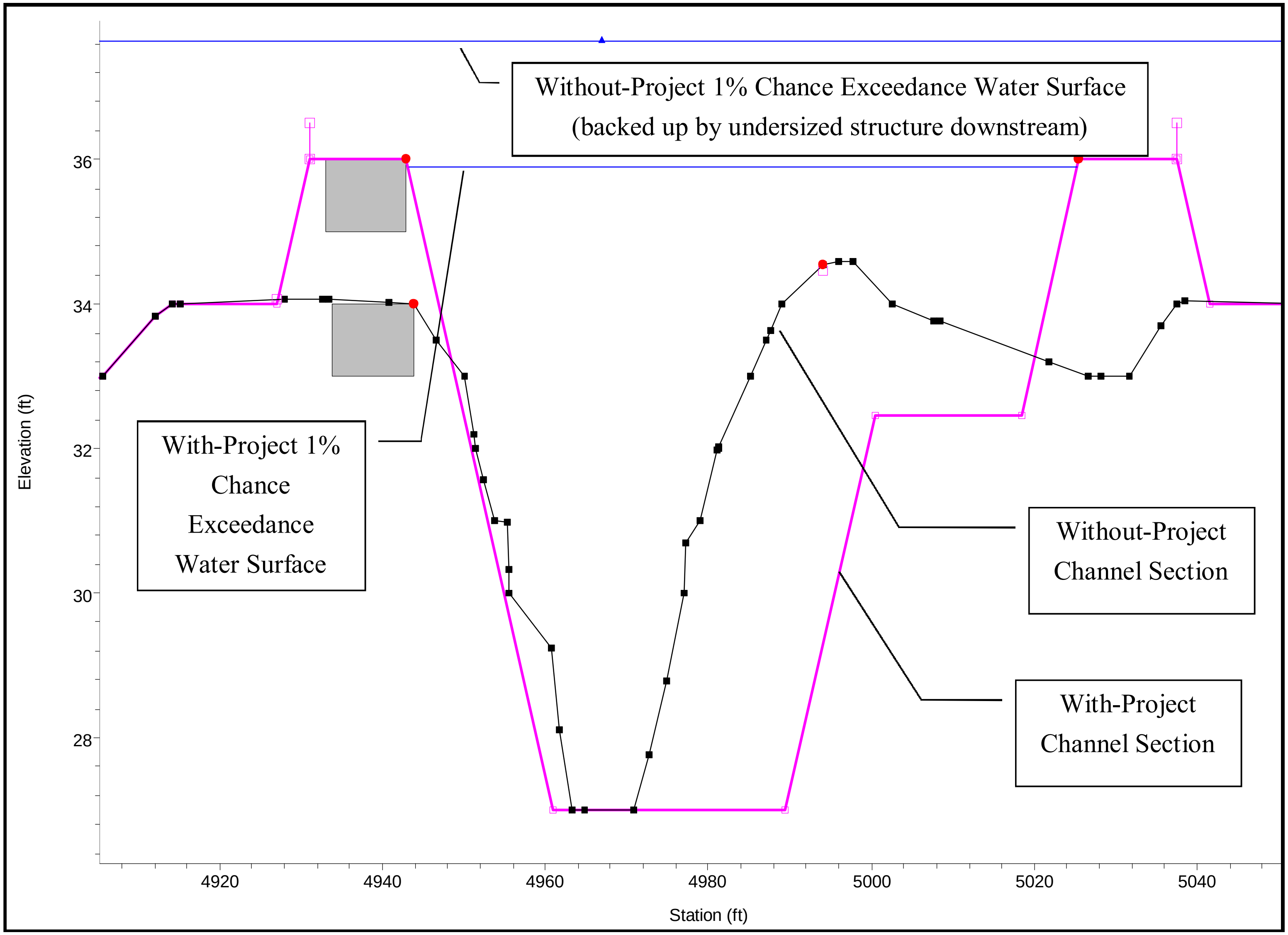

Figure 4-10 Typical D/S section, with- and without-project geometry for Alt. 2A

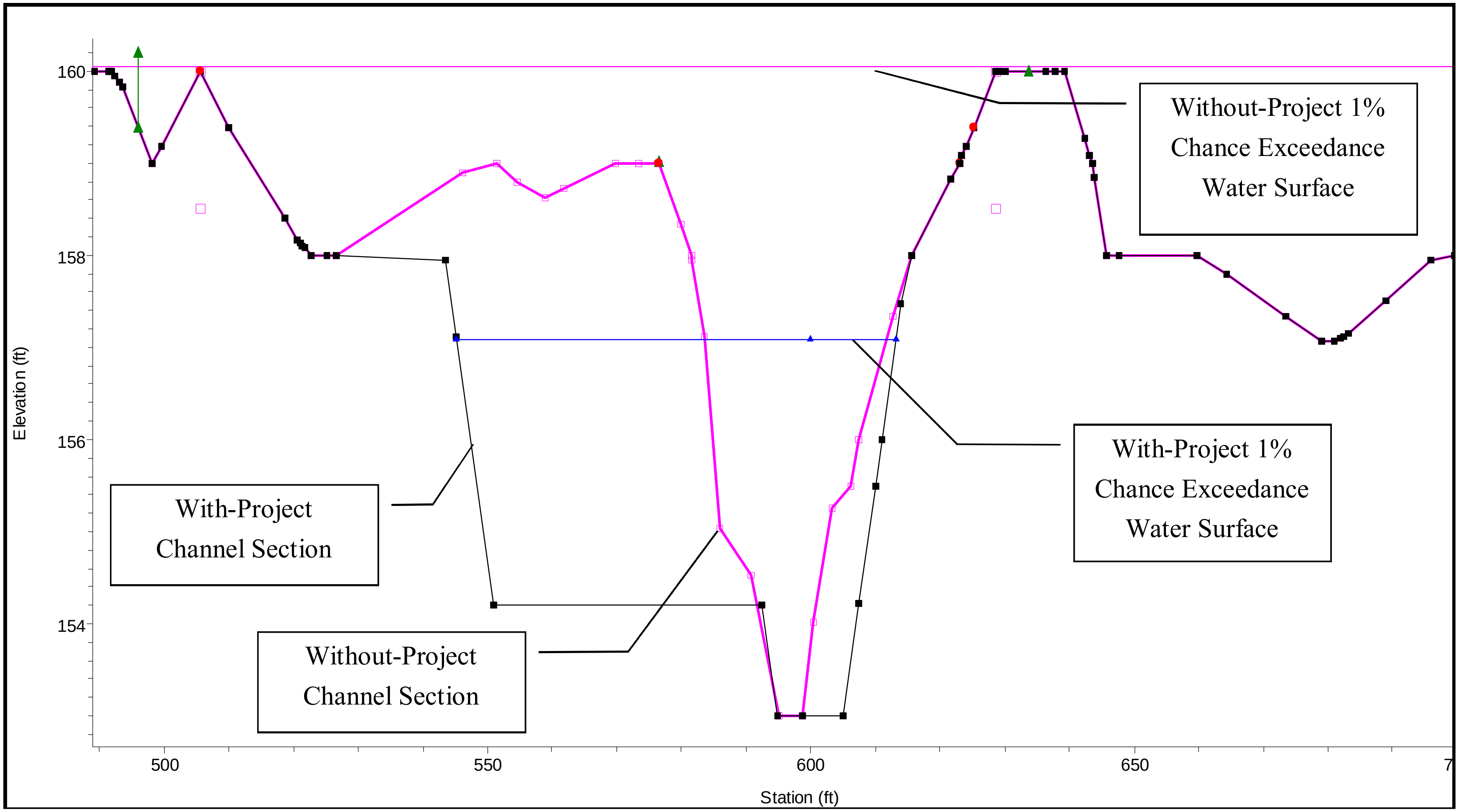

Figure 4-11 Typical U/S section, with- and without-project geometry for B Alternatives

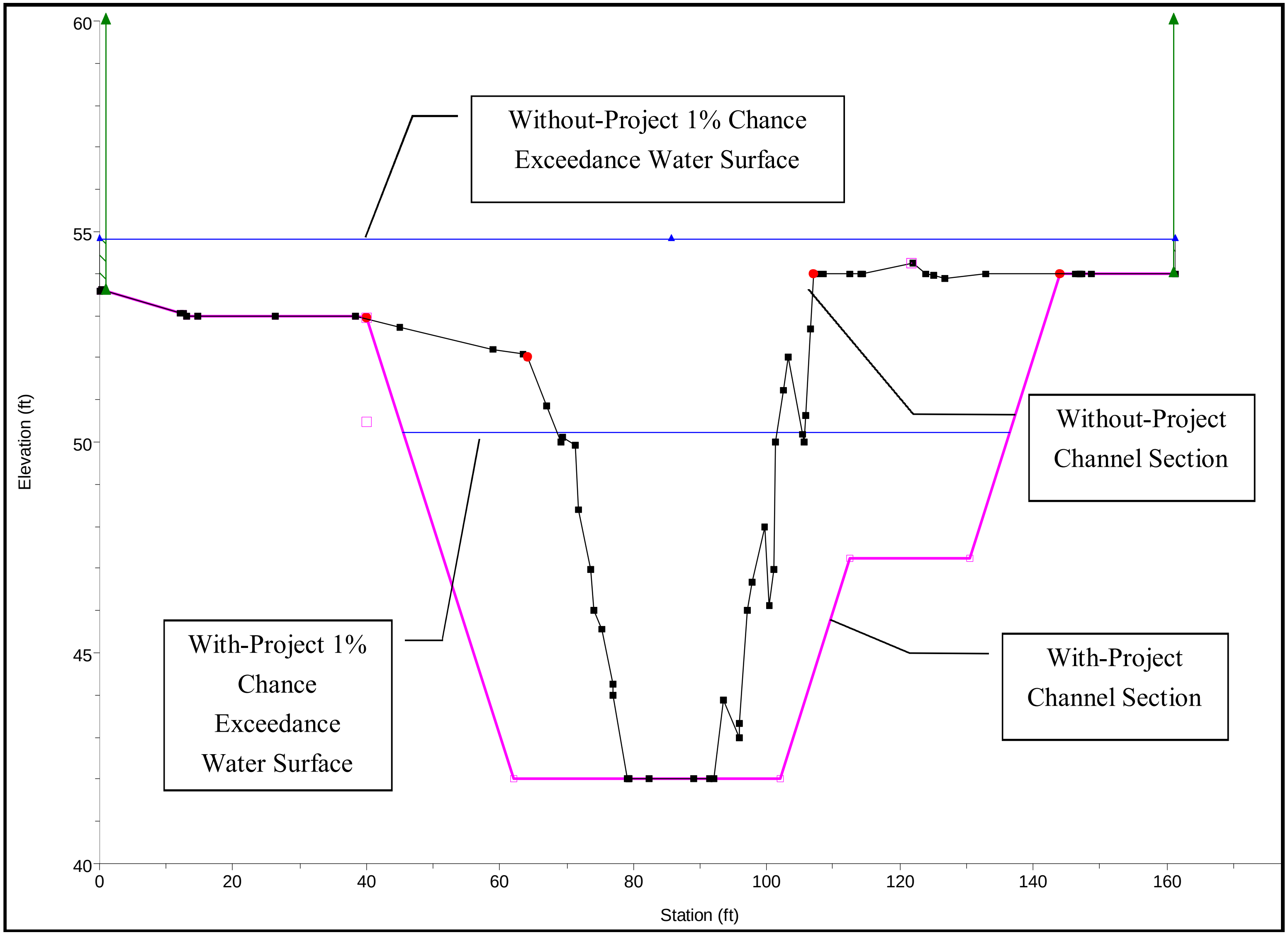

Figure 4-12 Typical D/S section, with- and without project geometry for Alt. 2B

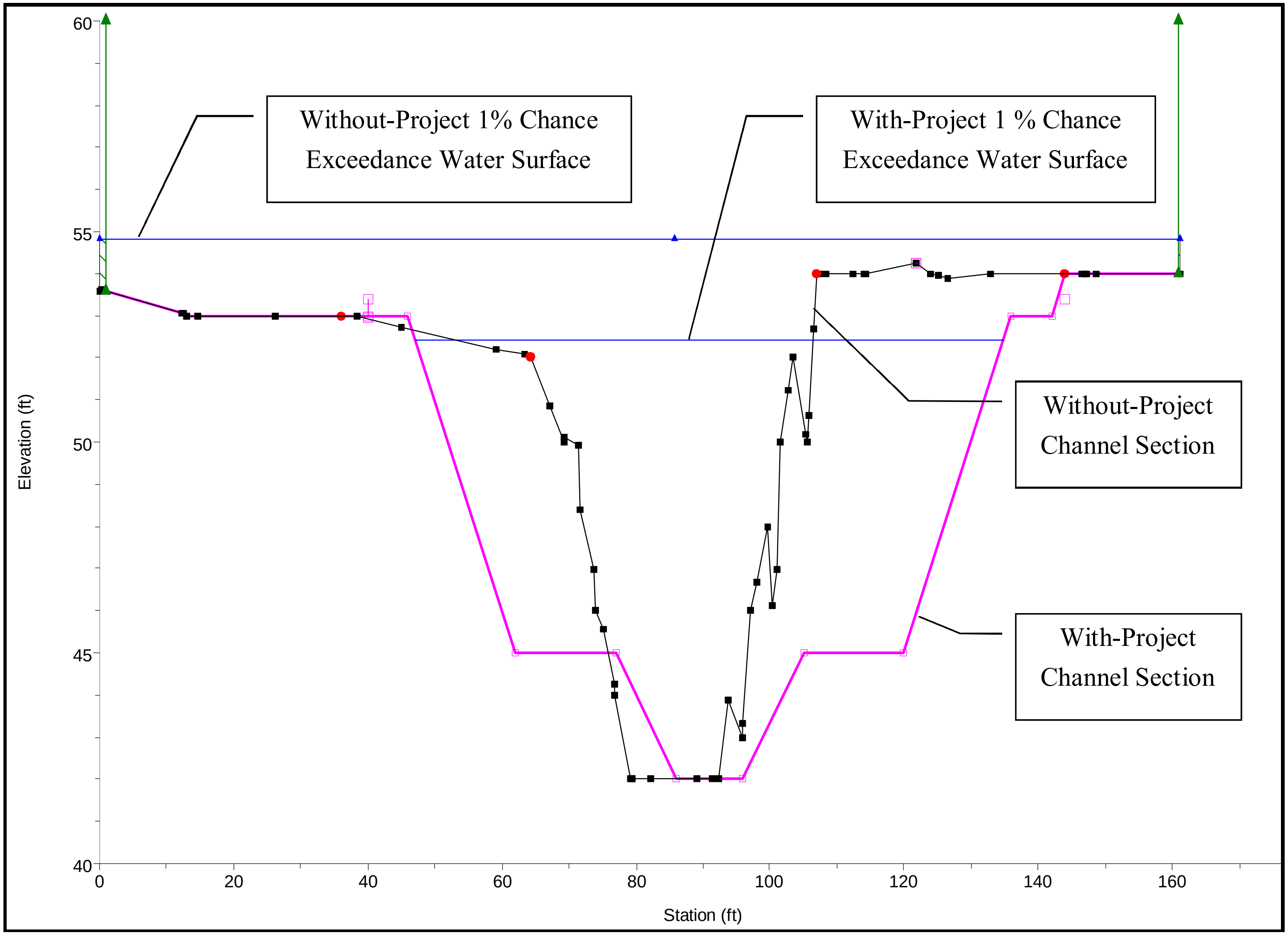

Figure 4-13 Typical D/S section, with- and without-project geometry for Alt. 3B

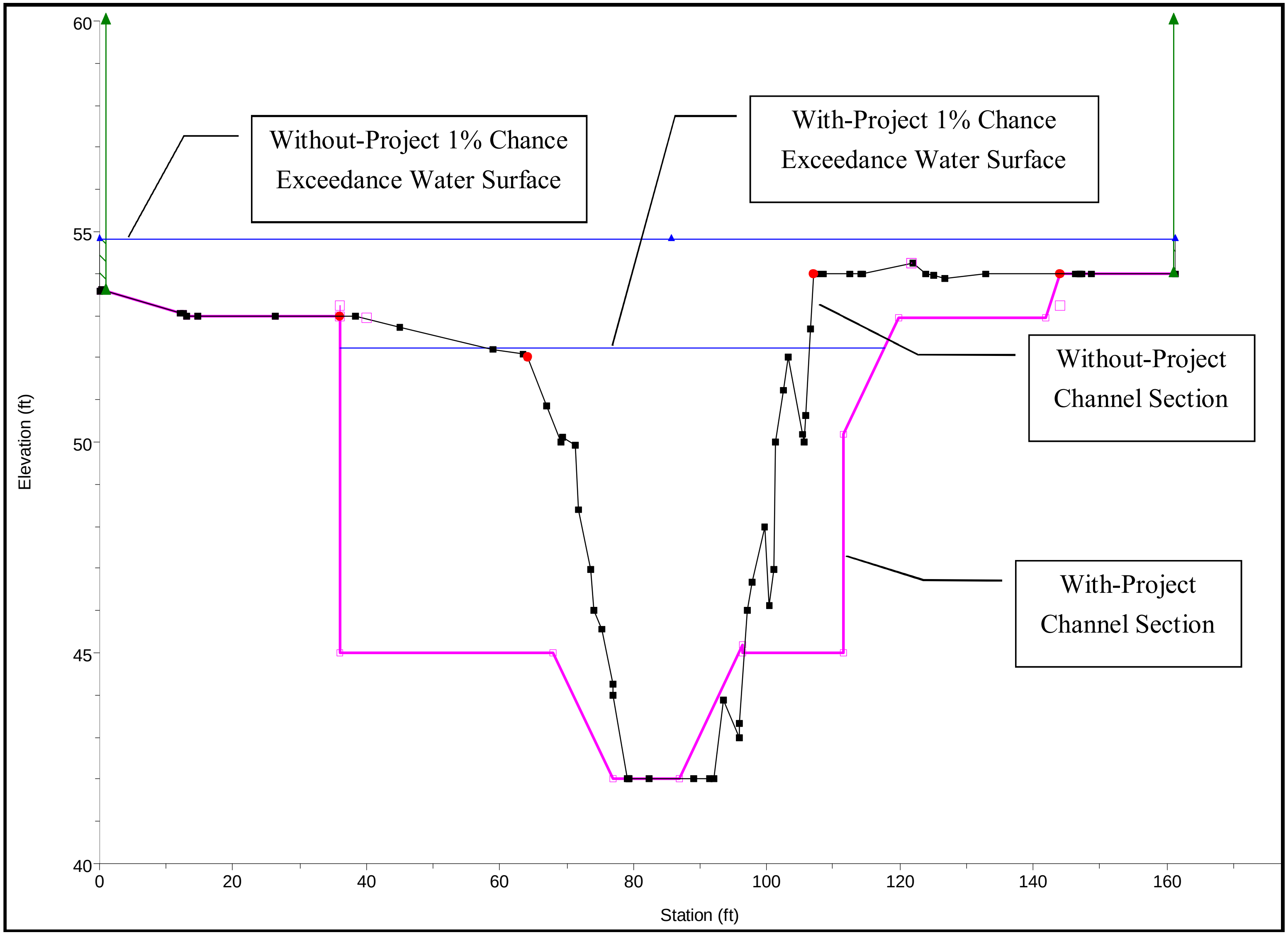

Figure 4-14 Typical D/S section, with- and without-project geometry for Alt. 4B

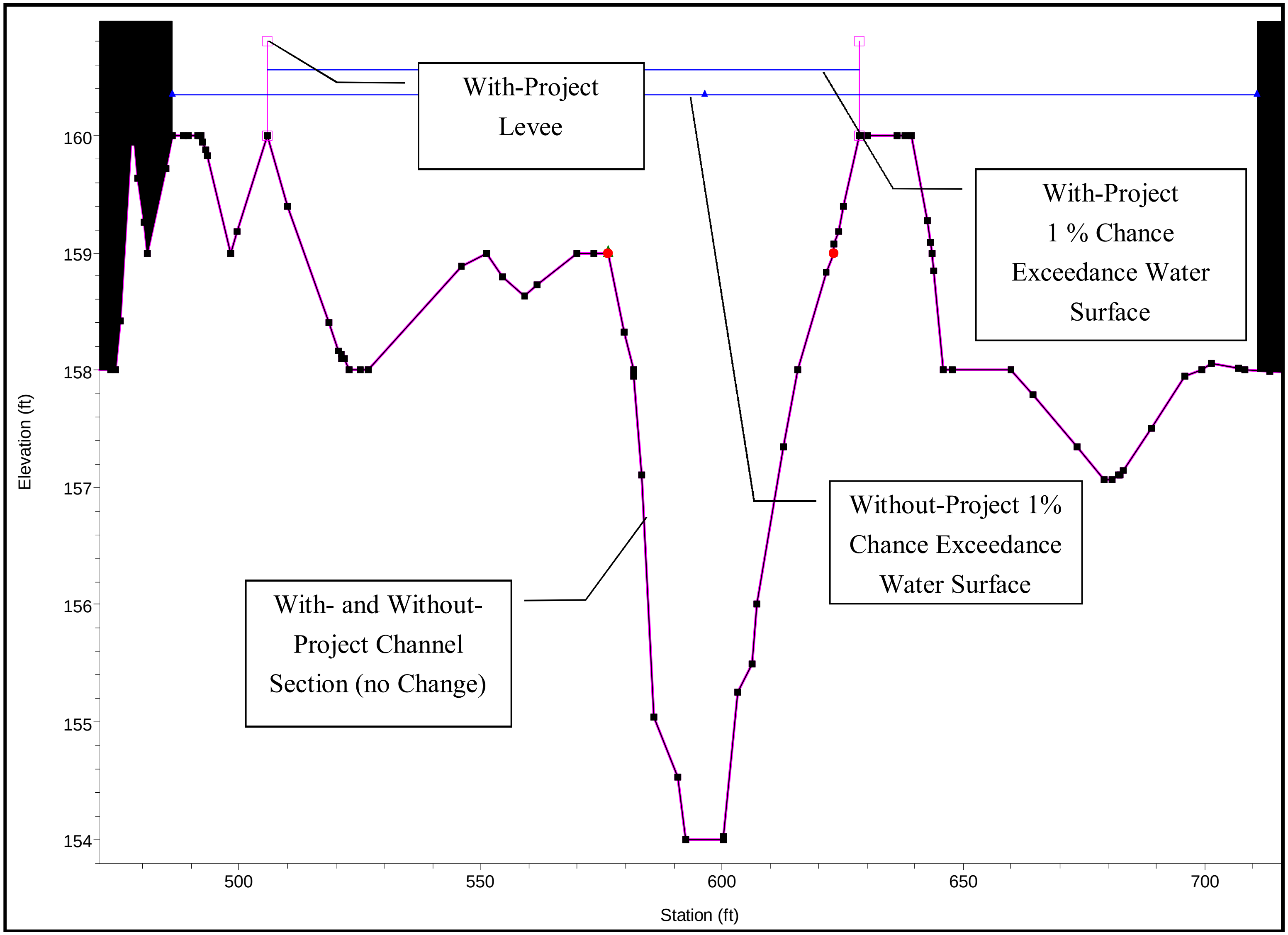

Figure 4-15 Typical U/S section, with- and without-project conditions for Alternative 5

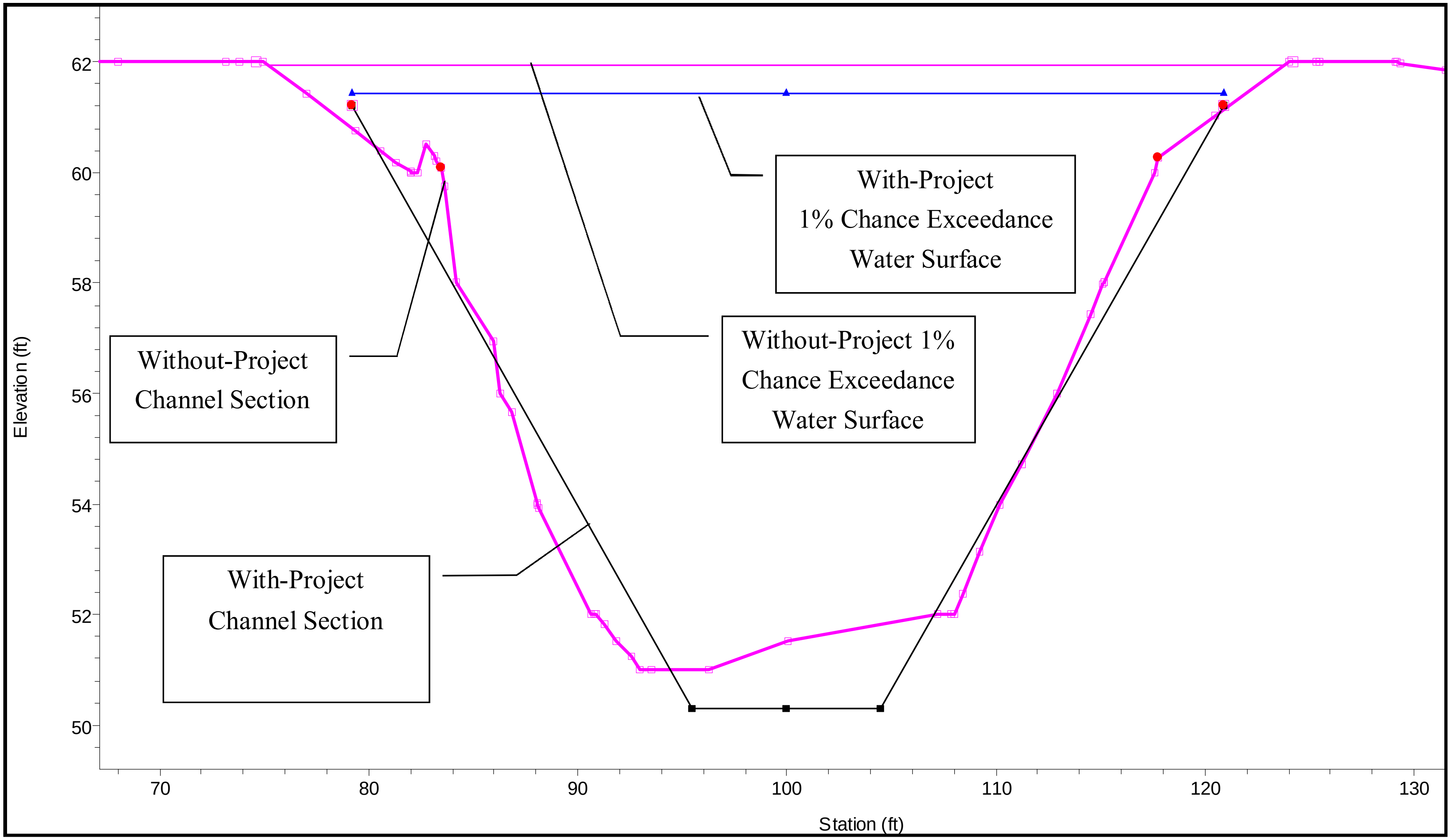

Figure 4-16 Typical D/S section, with- and without-project conditions for Alternative 5

Table 4-10 Summary of 1% Chance Exceedance Water Surface Elevations by Alternatives

Sta	Baseline		Alt 2A		Alt 2B		Alt 3B		Alt 4B		Alt 5	
	Vel	Depth	Vel	Depth	Vel	Depth	Vel	Depth	Vel	Depth	Vel	Depth
	fps	ft	fps	ft	fps	ft	fps	ft	fps	ft	fps	ft
36242	10.2	3.2	9.8	3.3	10.2	3.2	10.2	3.2	10.2	3.2	10.2	3.2
36126	9.7	4.7	8.7	2.4	9.7	4.7	9.7	4.7	9.7	4.7	9.7	4.7
36032	9.1	4.1	9.1	4.1	9.1	4.1	9.1	4.1	9.1	4.1	9.5	5.2
35589	8.4	3.6	8.4	3.6	8.4	3.6	8.4	3.6	8.4	3.6	10.8	3.6
35586	7.1	4.3	7.1	4.3	7.1	4.3	7.1	4.3	7.1	4.3	11.0	3.8
35476	10.4	3.4	10.4	3.4	10.4	3.4	10.4	3.4	10.4	3.4	7.2	1.6
35448	6.9	4.9	6.8	5.0	6.9	4.9	6.9	4.9	6.9	4.9	0.7	14.0
35418	10.7	4.3	10.8	4.2	10.7	4.3	10.7	4.3	10.7	4.3	0.7	14.4
35350	9.0	4.4	10.1	4.0	9.0	4.4	9.0	4.4	9.0	4.4	0.6	16.6
35285	7.2	6.5	8.9	5.7	7.2	6.5	7.2	6.5	7.2	6.5	1.5	7.2
35249	2.9	5.4	3.4	4.9	2.9	5.4	2.9	5.4	2.9	5.4	6.3	11.2
35191	8.7	2.0	7.1	9.8	8.7	2.0	8.7	2.0	8.7	2.0	8.1	11.9
Old Piedmont												
35139	14.8	6.8	12.2	5.7	14.8	6.8	14.8	6.8	14.8	6.8	13.8	5.9
35134	12.1	4.5	12.1	4.5	12.1	4.5	12.1	4.5	12.1	4.5	8.6	8.7
35132	14.1	6.1	13.6	5.8	14.1	6.1	14.1	6.1	14.1	6.1	8.6	8.7
35029	8.3	5.7	11.7	4.3	8.3	5.7	8.3	5.7	8.3	5.7	10.3	5.0
34989	8.6	6.3	8.0	5.8	8.6	6.3	8.6	6.3	8.6	6.3	12.0	4.5
34959	13.2	5.4	9.3	4.1	13.2	5.4	13.2	5.4	13.2	5.4	12.0	4.5
34909	9.6	5.9	9.9	5.3	9.6	5.9	9.6	5.9	9.6	5.9	12.0	4.5
34863	13.0	5.3	11.2	5.5	13.0	5.3	13.0	5.3	13.0	5.3	12.0	4.5
34779	12.0	4.5	10.0	4.9	12.0	4.5	12.0	4.5	12.0	4.5	12.1	4.5
34694	8.6	4.3	11.0	5.8	8.6	4.3	8.6	4.3	8.6	4.3	12.1	4.5
34566	7.2	4.0	6.1	6.6	7.2	4.0	7.2	4.0	7.2	4.0	12.1	4.5
34467	14.0	9.1	5.7	9.7	14.0	9.1	14.0	9.1	14.0	9.1	10.3	12.5
Piedmont-Cropley												
34041	7.5	3.5	12.1	4.5	7.5	3.5	7.5	3.5	7.5	3.5	7.5	3.5
34032	4.7	6.7	10.8	3.6	4.7	6.7	4.7	6.7	4.7	6.7	4.0	12.1
34010	2.6	7.6	2.9	8.0	2.6	7.6	2.6	7.6	2.6	7.6	2.8	8.4
33997	2.7	7.3	3.3	7.0	2.7	7.3	2.7	7.3	2.7	7.3	2.8	8.4
33966	2.8	7.3	3.9	6.3	2.8	7.3	2.8	7.3	2.8	7.3	3.1	7.1
33952	2.8	7.1	3.8	7.5	2.8	7.1	2.8	7.1	2.8	7.1	3.0	7.0
33942	3.8	5.2	6.1	5.9	3.8	5.2	3.8	5.2	3.8	5.2	4.3	4.9
33933	5.7	3.5	6.3	6.0	5.7	3.5	5.7	3.5	5.7	3.5	3.4	6.0
33904	9.0	2.5	7.2	5.6	9.0	2.5	9.0	2.5	9.0	2.5	4.6	7.2
33804	6.0	2.6	3.5	4.2	6.0	2.6	6.0	2.6	6.0	2.6	5.0	3.0
33773	5.0	3.3	7.4	2.3	5.0	3.3	5.0	3.3	5.0	3.3	5.0	3.7
33756	7.2	2.8	6.8	2.5	7.2	2.8	7.2	2.8	7.2	2.8	4.3	3.7
33485	6.3	2.6	6.8	3.1	6.3	2.6	6.3	2.6	6.3	2.6	10.5	3.4
33378	7.6	3.0	7.6	3.0	7.6	3.0	7.6	3.0	7.6	3.0	8.5	3.1
33207	6.4	2.4	8.0	2.4	6.4	2.4	6.4	2.4	6.4	2.4	6.4	2.4
33166	5.2	2.5	8.3	2.8	5.2	2.5	5.2	2.5	5.2	2.5	5.2	2.5
33136	7.8	1.9	9.5	2.8	7.8	1.9	7.8	1.9	7.8	1.9	7.9	1.9
32976	5.8	2.3	5.3	4.5	5.8	2.3	5.8	2.3	5.8	2.3	5.6	2.4
32889	6.4	2.1	5.6	4.3	6.4	2.1	6.4	2.1	6.4	2.1	8.8	3.6
32877	6.2	2.2	9.0	2.5	6.2	2.2	6.2	2.2	6.2	2.2	7.1	2.9
32753	9.3	2.7	7.2	3.4	9.3	2.7	9.3	2.7	9.3	2.7	6.2	3.5

Sta	Baseline		Alt 2A		Alt 2B		Alt 3B		Alt 4B		Alt 5	
	Vel	Depth	Vel	Depth	Vel	Depth	Vel	Depth	Vel	Depth	Vel	Depth
	fps	ft	fps	ft	fps	ft	fps	ft	fps	ft	fps	ft
32721	4.7	4.7	6.0	3.6	4.7	4.7	4.7	4.7	4.7	4.7	8.6	4.4
32659	9.0	3.9	5.2	4.6	9.0	3.9	9.0	3.9	9.0	3.9	8.4	4.1
32645	5.6	3.4	10.0	3.1	5.6	3.4	5.6	3.4	5.6	3.4	8.6	4.0
32631	5.7	3.3	7.4	3.3	5.7	3.3	5.7	3.3	5.7	3.3	8.6	4.1
32580	5.5	3.7	6.5	3.6	5.5	3.7	5.5	3.7	5.5	3.7	8.4	3.8
32436	4.9	2.6	8.2	2.4	4.9	2.6	4.9	2.6	4.9	2.6	8.6	3.1
32333	7.1	4.0	8.1	3.2	7.1	4.0	7.1	4.0	7.1	4.0	9.1	4.5
32208	5.3	3.9	8.4	2.9	5.3	3.9	5.3	3.9	5.3	3.9	7.3	3.9
32097	7.0	3.0	5.2	3.2	7.0	3.0	7.0	3.0	7.0	3.0	7.0	3.0
31969	7.7	2.3	9.3	2.7	7.7	2.3	7.7	2.3	7.7	2.3	7.7	2.3
31905	5.9	3.0	9.5	2.8	5.9	3.0	5.9	3.0	5.9	3.0	8.3	2.5
31716	8.3	3.7	5.7	3.8	8.3	3.7	8.3	3.7	8.3	3.7	5.2	4.7
31587	1.0	3.9	9.6	2.9	1.0	3.9	1.0	3.9	1.0	3.9	6.7	4.9
31571	5.7	4.0	5.8	4.0	5.7	4.0	5.7	4.0	5.7	4.0	9.0	4.8
31559	9.2	3.4	6.0	3.9	9.2	3.4	9.2	3.4	9.2	3.4	8.9	4.7
31440	6.2	2.7	6.0	3.0	6.2	2.7	6.2	2.7	6.2	2.7	6.3	3.1
31322	6.8	1.4	6.8	2.6	6.8	1.4	6.8	1.4	6.8	1.4	7.6	1.9
31168	3.0	2.9	8.5	2.7	3.0	2.9	3.0	2.9	3.0	2.9	3.7	3.0
31078	5.2	2.2	5.6	4.2	5.2	2.2	5.2	2.2	5.2	2.2	5.9	2.5
31026	5.4	2.7	5.3	3.9	5.4	2.7	5.4	2.7	5.4	2.7	5.4	2.7
30978	7.9	2.7	10.3	3.3	7.9	2.7	7.9	2.7	7.9	2.7	8.0	2.7
30965	7.1	2.8	7.2	2.9	7.1	2.8	7.1	2.8	7.1	2.8	7.1	2.9
30952	8.3	3.2	7.1	3.3	8.3	3.2	8.3	3.2	8.3	3.2	8.3	3.3
30910	7.2	4.8	7.4	3.2	7.2	4.8	7.2	4.8	7.2	4.8	7.2	4.8
30808	8.4	2.6	6.1	3.3	8.4	2.6	8.4	2.6	8.4	2.6	8.4	2.6
30731	6.2	3.1	6.8	3.1	6.2	3.1	6.2	3.1	6.2	3.1	6.2	3.1
30720	7.3	2.5	6.8	2.9	7.3	2.5	7.3	2.5	7.3	2.5	7.3	2.5
30701	8.2	2.3	7.2	2.9	8.2	2.3	8.2	2.3	8.2	2.3	8.2	2.3
30590	5.2	4.4	8.4	2.2	5.2	4.4	5.2	4.4	5.2	4.4	5.2	4.4
30478	8.3	2.7	5.3	3.9	8.3	2.7	8.3	2.7	8.3	2.7	7.1	2.4
30327	4.9	3.1	6.8	3.8	4.9	3.1	4.9	3.1	4.9	3.1	5.5	3.1
30324	6.0	2.8	5.0	4.1	6.0	2.8	6.0	2.8	6.0	2.8	5.7	3.0
30304	6.3	2.6	6.0	3.5	6.3	2.6	6.3	2.6	6.3	2.6	5.9	2.8
30195	5.2	2.5	9.2	2.6	5.2	2.5	5.2	2.5	5.2	2.5	5.7	2.6
30043	8.2	2.1	7.2	3.3	8.2	2.1	8.2	2.1	8.2	2.1	8.2	2.1
29983	5.1	3.2	6.8	3.4	5.1	3.2	5.1	3.2	5.1	3.2	5.1	3.2
29873	9.4	4.5	6.5	4.2	9.4	4.5	9.4	4.5	9.4	4.5	9.4	4.5
29744	7.0	4.2	5.4	3.1	7.0	4.2	7.0	4.2	7.0	4.2	8.0	3.9
29571	9.1	3.2	7.3	2.8	9.1	3.2	9.1	3.2	9.1	3.2	6.1	4.2
29433	6.2	3.6	5.2	3.7	6.2	3.6	6.2	3.6	6.2	3.6	9.5	3.9
29267	5.9	5.4	7.5	3.9	5.9	5.4	5.9	5.4	5.9	5.4	8.3	5.0
29231	3.9	6.3	3.9	7.1	3.9	6.3	3.9	6.3	3.9	6.3	4.6	6.2
29215	5.1	7.3	6.0	6.7	5.1	7.3	5.1	7.3	5.1	7.3	5.2	7.2
29199	14.1	6.1	14.1	6.2	14.1	6.1	14.1	6.1	14.1	6.1	14.1	6.1
29171	14.0	6.6	14.4	6.5	14.4	6.5	14.4	6.5	14.4	6.5	14.4	6.5
29093	8.5	5.4	10.1	5.1	9.5	5.3	10.1	5.1	9.5	5.3	11.6	5.1
28917	8.3	5.6	11.8	5.0	9.6	5.3	11.9	5.0	9.6	5.3	12.6	4.9
28770	5.6	5.9	8.3	4.7	6.4	5.2	8.2	4.7	6.4	5.2	3.6	13.0

Sta	Baseline		Alt 2A		Alt 2B		Alt 3B		Alt 4B		Alt 5	
	Vel	Depth	Vel	Depth	Vel	Depth	Vel	Depth	Vel	Depth	Vel	Depth
	fps	ft	fps	ft	fps	ft	fps	ft	fps	ft	fps	ft
28758	4.7	7.3	5.8	5.9	5.1	6.7	5.8	5.9	5.1	6.7	4.3	13.0
28749	5.1	5.7	8.5	4.3	5.8	5.0	7.5	3.9	5.8	5.0	4.3	13.0
28738	6.4	6.9	11.6	4.3	7.3	6.1	11.6	4.3	7.3	6.1	4.3	13.1
28699	4.0	4.7	9.4	2.8	4.9	3.8	9.4	2.8	4.9	3.8	4.2	13.3
28656	6.7	5.1	7.5	7.3	9.9	9.3	9.9	9.3	9.9	9.3	6.7	5.1
Morrill												
28528	13.3	5.5	11.6	7.1	13.3	5.5	13.3	5.5	13.3	5.5	13.3	5.5
28447	5.9	6.2	6.8	5.7	6.1	6.1	6.1	6.0	6.1	6.0	7.3	6.7
28307	6.1	5.6	7.9	4.8	6.5	5.4	6.6	5.4	6.6	5.4	7.0	6.8
28171	6.0	7.4	8.0	5.2	6.5	7.0	6.6	6.9	6.6	6.9	6.8	7.0
28025	4.3	6.7	6.4	3.8	4.6	6.3	4.7	6.2	4.7	6.2	6.5	7.1
27895	3.0	9.7	4.9	5.4	4.0	9.3	3.3	9.3	3.3	9.3	6.2	7.3
27705	4.1	9.6	5.8	6.4	4.4	9.3	4.5	9.2	4.5	9.2	6.0	7.6
27689	4.1	9.6	5.9	5.8	4.4	9.3	4.5	9.2	4.5	9.2	5.9	7.6
27675	4.1	9.7	5.7	6.4	4.4	9.3	4.5	9.2	4.5	9.2	5.9	7.6
27658	3.7	7.6	5.2	5.7	3.9	7.3	4.0	7.2	4.0	7.2	5.9	7.6
27642	6.0	6.9	7.9	11.6	10.9	4.8	11.4	10.1	11.4	10.1	7.7	11.3
Cropley												
27499	14.5	8.0	9.7	9.4	14.5	8.0	14.5	8.0	14.5	8.0	14.5	8.0
27481	9.3	6.3	9.4	6.3	9.4	6.3	9.3	6.3	9.3	6.3	9.4	6.3
27459	13.0	5.3	13.0	5.3	13.0	5.3	13.0	5.3	13.0	5.3	13.0	5.3
27380	12.7	5.0	12.7	5.0	12.7	5.0	12.7	5.0	12.7	5.0	12.7	5.0
27108	12.0	4.5	12.0	4.5	12.0	4.5	12.0	4.5	12.0	4.5	12.0	4.5
26889	13.3	5.5	13.3	5.5	13.3	5.5	13.3	5.5	13.3	5.5	13.3	5.5
26695	12.4	5.7	12.4	5.7	12.4	5.7	12.4	5.7	12.4	5.7	12.4	5.7
26577	12.2	4.6	12.2	4.6	12.2	4.6	12.2	4.6	12.2	4.6	12.2	4.6
26419	13.2	5.4	13.2	5.4	13.2	5.4	13.2	5.4	13.2	5.4	13.1	5.4
26288	13.0	5.3	13.0	5.3	13.0	5.3	13.0	5.3	13.0	5.3	13.0	5.3
26123	13.0	5.2	13.0	5.2	13.0	5.2	13.0	5.2	13.0	5.2	13.0	5.2
25955	13.4	5.6	13.4	5.6	13.4	5.6	13.4	5.6	13.4	5.6	13.4	5.6
25798	12.8	5.2	12.8	5.2	12.8	5.2	12.8	5.2	12.8	5.2	12.8	5.2
25744	12.9	5.2	12.9	5.2	12.9	5.2	12.9	5.2	12.9	5.2	12.9	5.2
25719	6.8	6.4	6.8	6.4	6.8	6.4	6.8	6.4	6.8	6.4	8.6	6.2
25705	6.0	6.4	6.0	6.4	6.0	6.4	6.0	6.4	6.0	6.4	7.5	6.2
25688	5.2	7.0	5.2	7.0	5.2	7.0	5.2	7.0	5.2	7.0	6.5	6.3
I-680												
25296	5.4	5.5	5.4	5.5	5.4	5.5	5.4	5.5	5.4	5.5	5.9	5.2
25245	6.2	6.6	6.3	6.6	6.3	6.6	6.3	6.6	6.2	6.6	6.9	6.0
25155	6.4	5.1	6.4	5.1	6.4	5.1	6.4	5.1	6.4	5.1	7.4	5.2
24997	7.5	5.3	7.5	5.3	7.5	5.3	7.5	5.3	7.5	5.3	8.4	6.1
24886	7.3	5.6	7.3	5.6	7.3	5.6	7.3	5.6	7.3	5.6	8.7	6.0
24791	7.1	5.2	7.1	5.2	7.1	5.2	7.1	5.2	7.1	5.2	9.2	5.8
24694	8.2	4.6	8.3	4.6	8.3	4.6	8.3	4.6	8.3	4.6	11.4	5.1
24171	9.2	5.6	9.1	5.7	9.2	5.7	9.2	5.7	9.2	5.7	8.7	5.9
24079	10.7	5.0	10.8	5.0	10.8	5.0	10.8	5.0	10.8	5.0	8.6	5.9
23986	8.9	5.3	10.7	3.6	10.5	3.4	8.6	4.3	7.3	4.4	8.6	5.9
23889	7.8	4.7	10.2	3.3	10.2	3.2	8.6	4.3	7.3	4.4	8.5	5.9
23786	6.3	4.4	8.3	3.5	8.5	3.4	8.6	4.3	7.2	4.4	8.4	6.0

Sta	Baseline		Alt 2A		Alt 2B		Alt 3B		Alt 4B		Alt 5	
	Vel	Depth	Vel	Depth	Vel	Depth	Vel	Depth	Vel	Depth	Vel	Depth
	fps	ft	fps	ft	fps	ft	fps	ft	fps	ft	fps	ft
23710	4.8	5.0	6.9	3.7	8.4	3.4	8.6	4.3	7.2	4.4	8.3	6.1
23610	7.1	5.0	6.2	4.0	8.4	3.4	8.6	4.3	7.2	4.4	8.1	6.2
23522	6.3	5.7	5.8	4.3	8.2	3.5	8.6	4.3	7.2	4.4	7.9	6.3
23413	7.2	5.0	5.6	4.4	7.9	3.6	8.6	4.3	7.2	4.4	7.7	6.5
23326	6.5	4.8	5.3	4.6	7.5	3.7	8.6	4.3	7.2	4.4	7.5	6.7
23185	5.4	4.8	5.2	4.4	6.7	4.1	8.6	4.3	7.2	4.5	7.1	7.1
23062	4.6	6.0	4.4	5.4	5.9	4.5	8.6	4.3	7.2	4.5	4.8	8.6
22951	5.5	5.6	4.6	5.2	5.3	4.9	8.5	4.3	7.1	4.5	4.1	10.1
22865	10.5	4.9	4.6	5.1	4.9	5.2	8.5	4.4	7.1	4.5	4.1	10.0
22806	9.4	4.7	4.6	5.1	4.7	5.4	8.4	4.4	7.0	4.6	4.1	10.1
22748	9.0	4.5	4.6	5.2	8.3	4.2	8.3	4.4	7.0	4.6	4.1	10.1
22693	8.5	5.1	4.6	5.2	8.1	4.3	8.2	4.5	6.9	4.7	4.1	10.2
22603	9.9	5.6	4.7	5.5	7.6	4.5	8.0	4.6	6.7	4.8	4.0	10.4
22274	3.1	7.5	2.8	7.2	5.7	5.5	6.4	5.6	5.7	5.7	3.9	10.5
22117	7.6	6.8	3.2	7.4	5.0	6.0	5.7	6.3	5.2	6.2	3.9	10.6
21883	5.3	7.2	3.0	7.9	4.2	6.8	4.7	7.3	4.5	7.2	3.9	10.7
21873	4.9	7.5	3.0	8.0	4.1	6.8	4.7	7.4	4.4	7.3	3.9	10.7
21864	4.3	7.5	2.9	8.1	4.1	6.8	4.7	7.4	4.4	7.3	3.9	10.8
21852	3.6	4.7	2.8	8.2	4.1	6.9	4.6	7.5	4.4	7.4	3.9	10.8
21844	3.7	4.8	2.8	8.3	4.1	6.9	4.6	7.5	4.3	7.4	3.9	10.8
21832	4.4	4.7	2.8	8.4	4.0	7.0	4.6	7.6	4.3	7.4	3.9	10.8
21821	7.3	4.8	6.8	12.6	3.7	8.7	9.2	6.5	6.6	7.2	4.4	11.8
Montague Expressway												
21800	7.9	7.2	7.1	12.1	4.9	8.3	10.1	6.0	6.8	7.2	5.5	9.4
21657	4.3	8.8	4.8	7.2	8.4	6.3	7.3	6.1	5.6	7.2	8.5	7.7
21646	4.2	8.0	4.9	7.0	8.4	6.3	7.2	6.2	5.6	7.2	8.5	7.7
21634	4.7	7.0	5.8	6.6	8.4	6.3	7.2	6.2	5.5	7.2	8.4	7.7
21623	6.6	6.7	5.8	6.7	8.4	6.3	7.2	6.2	5.5	7.3	8.4	7.7
21601	9.4	7.3	5.8	6.6	8.3	6.4	7.1	6.2	5.5	7.3	8.4	7.7
21314	7.9	8.5	5.8	6.5	7.4	6.9	6.2	7.1	4.8	8.3	8.2	7.9
21276	6.1	8.6	5.6	6.6	7.3	7.0	6.0	7.2	4.8	8.4	8.2	7.9
21270	5.7	8.6	4.8	8.1	4.5	9.2	5.2	7.2	4.0	9.9	8.2	7.9
UPRR Trestle												
21226	6.9	7.7	6.2	6.6	4.8	8.9	6.5	6.3	5.4	7.4	9.8	6.9
21219	9.7	8.4	9.8	5.9	9.9	5.8	8.6	5.3	7.4	5.4	9.8	6.9
21203	12.3	6.4	12.8	5.1	9.9	5.8	8.6	5.3	7.4	5.4	9.8	6.9
21050	9.8	6.6	9.8	5.5	9.9	5.8	8.6	5.3	7.4	5.4	9.8	6.9
20823	9.7	7.7	9.8	5.5	9.9	5.8	8.6	5.3	7.3	5.5	9.7	6.9
20595	9.4	7.8	9.7	5.6	9.8	5.9	8.4	5.4	7.2	5.6	9.7	7.0
20368	10.5	7.0	12.5	4.8	9.7	5.9	8.0	5.6	7.0	5.7	9.6	7.0
20131	4.4	8.2	7.5	5.5	12.8	5.1	7.4	6.0	6.6	6.1	9.5	7.1
19901	8.9	6.8	6.7	6.0	6.6	5.3	6.7	6.6	6.2	6.5	9.3	7.2
19676	11.0	6.6	5.9	6.5	5.7	5.9	6.0	7.3	5.7	7.1	9.1	7.4
19413	5.2	6.9	4.5	7.7	4.9	6.6	6.5	6.8	5.1	7.9	3.7	12.1
19400	5.9	7.4	4.5	7.7	4.9	6.6	5.3	8.0	5.0	7.9	3.6	12.3
19390	5.5	13.9	7.1	10.8	7.6	9.2	7.4	9.6	7.3	9.6	3.6	12.4
UPRR Triple Box												
19296	5.8	12.7	10.5	7.1	10.4	6.8	8.7	8.0	8.7	8.1	4.5	10.0

Sta	Baseline		Alt 2A		Alt 2B		Alt 3B		Alt 4B		Alt 5	
	Vel fps	Depth ft	Vel fps	Depth ft	Vel fps	Depth ft	Vel fps	Depth ft	Vel fps	Depth ft	Vel fps	Depth ft
19285	4.9	7.7	6.8	5.8	7.3	4.8	7.3	6.1	6.5	6.2	4.4	10.0
19268	4.0	7.1	6.6	5.7	7.3	4.8	7.1	6.2	6.5	6.2	4.4	10.1
19244	4.2	8.3	7.1	5.7	7.3	4.8	7.1	6.3	6.5	6.2	4.4	10.2
19234	4.2	8.3	7.2	5.7	7.3	4.8	7.1	6.3	6.5	6.2	4.4	10.2
19184	7.0	6.8	7.4	5.6	7.3	4.8	7.1	6.3	6.5	6.1	4.3	10.3
19172	7.4	6.3	7.5	5.6	7.3	4.8	7.1	6.3	6.5	6.1	4.3	10.4
19158	7.6	8.0	7.5	5.5	7.3	4.8	7.1	6.3	6.5	6.1	4.3	10.4
19083	8.9	4.9	8.0	5.3	7.3	4.8	7.1	6.3	6.6	6.1	7.7	8.8
18904	5.7	8.0	7.0	5.8	7.4	4.7	7.1	6.3	6.8	5.9	7.2	9.4
18881	5.0	6.6	6.9	5.8	6.5	5.0	6.7	5.3	6.8	5.9	7.2	9.5
Ames Ave												
18805	7.9	5.8	6.6	5.5	6.3	4.8	7.9	4.7	7.4	5.4	12.0	6.0
18774	10.5	6.9	6.8	5.5	8.9	3.8	8.7	5.2	7.4	5.4	11.0	6.3
18553	10.5	5.1	10.5	3.9	8.9	3.8	8.6	5.3	7.4	5.4	10.8	6.5
18259	6.8	4.9	9.0	4.4	8.8	3.8	8.5	5.3	7.4	5.4	10.2	6.9
18045	2.9	7.1	8.8	4.5	8.8	3.9	8.4	5.4	7.3	5.5	9.5	7.4
17811	2.0	8.6	9.5	4.2	8.5	4.0	8.0	5.6	7.2	5.6	8.6	8.1
17602	2.9	8.3	9.5	4.2	7.7	4.3	7.4	6.0	7.0	5.7	7.8	8.9
17571	7.7	9.9	7.9	5.0	7.0	5.5	6.1	6.1	6.1	6.5	6.9	9.5
Yosemite Dr												
17470	8.6	5.9	9.1	4.5	7.6	5.3	7.5	5.4	6.8	6.3	9.0	7.7
17448	6.5	5.5	8.4	3.7	7.6	4.1	7.8	6.1	6.8	6.3	9.0	7.8
17427	11.5	5.7	8.3	3.7	7.5	4.1	7.8	6.1	6.7	6.3	8.9	7.8
17281	9.1	3.4	7.0	4.3	7.3	4.2	7.5	6.3	6.6	6.5	8.6	8.0
16924	5.4	5.0	5.3	5.4	6.1	4.9	6.6	7.1	5.9	7.2	7.9	8.8
16654	5.1	6.3	6.3	5.9	8.8	4.6	9.4	6.7	8.2	7.0	12.4	7.5
16437	4.3	6.4	5.7	6.3	8.8	4.6	9.4	6.7	8.1	7.1	10.4	8.4
16139	6.3	6.9	6.9	6.1	8.6	4.7	9.3	6.8	7.9	7.2	10.1	8.6
15928	3.1	7.2	3.8	6.7	8.4	4.8	9.2	6.8	7.8	7.4	9.8	8.9
15665	3.0	8.1	6.6	6.8	7.8	5.1	8.9	7.0	7.5	7.7	9.4	9.3
15398	3.9	7.1	5.6	7.0	7.0	5.6	8.4	7.4	7.1	8.1	8.9	9.8
15156	3.9	8.2	4.5	8.1	6.1	6.3	7.8	7.8	6.7	8.6	8.4	10.4
14944	6.0	7.5	4.4	8.7	5.5	7.0	7.3	8.3	6.3	9.1	8.0	10.9
14685	6.1	8.3	3.7	9.4	4.8	7.9	6.6	9.0	5.8	9.8	7.5	11.7
14467	7.9	7.5	4.7	9.6	5.8	8.5	8.4	9.1	7.4	10.0	8.5	13.2
14422	3.8	11.1	3.8	11.2	5.9	7.6	5.1	8.7	7.2	10.1	5.5	14.2
Los Coches St.												
14350	4.4	9.4	5.2	7.5	6.1	7.4	5.4	8.3	8.3	8.8	6.3	12.3
14179	4.2	7.7	5.0	8.9	5.2	8.5	7.9	9.7	7.4	9.9	6.3	12.3
14121	8.6	7.0	4.9	9.1	4.6	9.5	7.0	10.6	6.8	10.8	6.3	12.3
13937	7.4	11.5	4.0	10.6	4.0	10.8	6.1	11.8	6.2	11.9	6.2	12.4
13887	7.0	12.3	3.8	10.9	4.3	11.5	4.1	11.9	6.0	12.2	7.0	12.3
Calaveras Blvd												
13741	7.8	9.1	6.9	6.7	4.4	10.8	4.2	11.1	6.4	11.4	7.9	9.1
13724	6.2	11.6	6.9	6.7	3.4	11.2	6.6	11.2	6.7	11.0	6.5	11.1
13661	3.8	8.4	6.8	6.0	3.4	11.4	6.6	11.1	6.7	10.9	3.9	9.1
13585	3.4	9.5	6.2	6.2	3.3	11.6	6.7	11.0	6.8	10.8	3.8	8.4
13509	4.6	10.8	5.5	6.7	4.6	10.9	4.7	10.8	4.6	10.8	3.4	9.5

CHAPTER 5: FINAL ARRAY OF PROJECT ALTERNATIVES

The final array of project alternatives were analyzed using the revised GRR HEC-RAS unsteady model, described in Section 2.2. Four alternatives were simulated using the revised GRR methodology models. Project features including the hydraulic structure capacities and top of bank/levee elevations for Alternatives 2B and 4 from the preliminary array of alternatives were revised to meet the requirements for FEMA certification using risk and uncertainty principles per Engineering Circular 1110-2-6067, *Certification of Levee Systems for the National Flood Insurance Program* (USACE 2008a) and were based on future improvements by the SCVWD upstream of I-680 constructed on the Berryessa Creek. Alternative 2A was revised to pass the 1% chance exceedance event using the revised GRR unsteady HEC-RAS modeling. No changes were made on project features for Alternative 5. The Berryessa Creek reach upstream of I-680 was removed from each alternative and the hydrologic inputs were developed to allow for unsteady runs to be made. The resulting alternatives are designated as 2A/d, 2B/d, and 4/d to indicate that they only include project features for Berryessa Creek downstream of I-680.

Alternatives 2B/d and 4/d were divided into five sub reaches with representative index cross section assigned to each reach. The Corps HEC-FDA program version 1.2.5a was used to determine the conditional non-exceedance probability (CNP). The hydraulic and hydrologic data developed for the GRR were used as inputs, along with the top of levee elevations to determine the CNP for each reach. Each reach was analyzed to determine if a minimum CNP of 90% for the 1% chance exceedance event (discharge based on future improvements by the SCVWD upstream of I-680) was achieved for entrenched channels. Based on the CNP results the alternatives were refined as needed and the process repeated until the desired minimum CNP of 90% was reached or exceeded (USACE 2008a).

5.1 Alternative descriptions

The alternatives evaluated include the No-action alternative and four project alternatives. Following is a list of features included with each alternative:

Alternative 1 (No Action) Without-project condition, assuming routine maintenance.

Alternative 2A/d (Incised Trapezoidal Channel). The alternative was designed assuming no project upstream of I-680, locally or federally developed, is in place. The primary characteristics of the alternative are as follows:

- Earthen trapezoidal section with varying bottom width and 2H:1V side slopes with a moderate level of containment
- Access road intermittently along top of bank or within channel at approximate level of 4% chance exceedance event
- Cellular bank stabilization with riprap toe protection throughout
- Levees with 2H:1V side slopes and 12’ top width in limited areas, with floodwalls on levees as required

- Montague Expressway, Ames Avenue, Yosemite Avenue, Los Coches Avenue, and Calaveras Boulevard bridges to be modified
- UPRR trestle bridge to be replaced

Alternative 2B/d (Incised Trapezoidal Channel). The alternative was designed assuming a bypass structure, to be developed and constructed along Berryessa Creek upstream of I-680 separately by the SCVWD as a locally funded project, is in place. The bypass will route high flows around the Greenbelt reach reducing flooding in the upper Berryessa watershed. The primary characteristics of the alternative are as follows:

- Earthen trapezoidal section with varying bottom width and 2H:1V side slopes with a FEMA-certifiable level of containment
- Access road intermittently along top of bank or within channel at approximate level of the 10 to 4% chance exceedance event with varying designed level of the maintenance road to suit local maintenance needs
- Cellular bank stabilization with riprap toe protection throughout
- Levees as required with 2H:1V side slopes and 12' top width
- Concrete floodwalls on levees where required
- Montague Expressway, UPRR trestle, Los Coches Avenue, and Calaveras Boulevard bridges to be replaced
- UPRR triple box culvert to be replaced
- Ames Avenue and Yosemite Avenue bridges to be modified.

Alternative 4/d (Walled Trapezoidal Channel). The alternative was designed assuming a bypass structure, to be developed and constructed along Berryessa Creek upstream of I-680 separately by the SCVWD as a locally funded project, is in place. The bypass will route high flows around the Greenbelt reach reducing flooding in the upper Berryessa watershed. The primary characteristics of the alternative are as follows:

- 10' bottom width earthen low-flow channel with 3H:1V side slopes, 3' deep with a FEMA-certifiable level of containment
- Two vegetated floodplain benches, 32' wide on the left bank, and 10' wide on the right bank
- Vertical concrete retaining walls bounding the benches
- Access road location varies along the top of one or both banks or within channel
- Floodwall extensions as required to contain flows
- Montague Expressway, UPRR timber bridge, Los Coches Avenue, and Calaveras Boulevard bridges to be replaced
- UPRR triple box culvert replaced
- Ames Avenue and Yosemite Avenue bridges to be modified

Alternative 5 (Authorized Plan). Alternative 5 is a single-purpose flood risk management project that includes mitigation of adverse effects as authorized by Congress in 1990 as the Berryessa Creek Project. Alternative 5 begins 600 feet upstream of the Old Piedmont Road and extends to 50 feet downstream of Calaveras Boulevard Bridge. The primary characteristics of the alternative are as follows:

- 500- by 160-foot reinforced-concrete-walled sedimentation basin at upstream end of the Authorized Project transitioning into a new box culvert under Old Piedmont Road
- Trapezoidal concrete-lined channel would be constructed with a bottom width of 8 feet and 2:1 (H:V) bank slopes from Old Piedmont Road to Piedmont Road/Cropley Avenue with service road along the east bank maintained, and with the riparian vegetation along the west bank retained as much as possible.
- Existing 400-foot-long box culvert under the Piedmont Road/Cropley Avenue intersection would be retained
- Existing debris basin downstream of Cropley Avenue would be enlarged and lined with concrete walls to function as a secondary sedimentation basin.
- Existing channel throughout the greenbelt area would be retained as much as possible and the existing levees would be raised to contain the design flood
- Transition area at the downstream end of the greenbelt (approximately 600 feet upstream of Morrill Avenue) leading into trapezoidal concrete-lined channel
- Trapezoidal Concrete channel from transition area until joining the existing concrete-lined channel downstream of Cropley Avenue
- Trapezoidal concrete-lined channel from end of existing concrete-lined channel at I-680 to Calaveras Boulevard
- Rock transition below Calaveras Boulevard to transition flows from the concrete channel into the existing earth-bottomed channel

Bridge and culvert modification and replacement scenarios are generally consistent between Alternatives 2B/d and 4/d. The alternatives differ only in the configuration of the channel reaches between the structures. Alternative 5 is based on the Authorized Project as authorized by Congress in 1990. Plan views and typical sections showing the overall configuration of Alternatives 2A/d, 2B/d and 4/d are presented in *Appendix B, Part IV: Design and Cost of Alternatives*.

5.2 Model Input

5.2.1 Discharge

The Revised Berryessa Creek HEC-RAS model requires hydrographs representing various inflows to the Berryessa Creek Channel. The inflow hydrographs to Berryessa Creek downstream of the I-680 culvert consists of local subarea runoff and tributary creeks. The upstream inflow hydrograph to the HEC-RAS model is the outflow from the I-680 culvert. The outflow used to size Alternative 2A/d was the same hydrograph developed from the Upper Berryessa Creek FLO-2D model for the without-project conditions runs as described in Section 2.2. Alternative 2B/d and 4/d were sized assuming bypass system is constructed by the local sponsor upstream of I-680. The inflow hydrograph at I-680 was therefore developed using a different methodology than for the without-project conditions using the I-680 culvert outflow hydrograph developed from the SCVWD Bypass HEC-HMS model (SCVWD 2011a, 2011b). Economic benefits for Alternatives 2B/d and 4/d were then derived using the without-project conditions as described in Section 2.2. A separate Upper Berryessa Creek FLO-2D model was created for Alternative 5 to model the portions of the alternative in the Upper Berryessa Reach (upstream of I-680). The Alternative 5 Upper Berryessa FLO-2D model is documented in *Appendix B, Part II: Without-Project Floodplain Development.*

5.2.2 Local and Tributary Inflow Hydrographs

The final array of alternatives includes two sets of local and tributary inflow conditions. Alternatives 2A/d and 5 were run assuming no future improvements are implemented on the Berryessa Creek system upstream of I-680. Alternatives 2B/d and 4/d were designed assuming that future improvements planned by the SCVWD are constructed in the Berryessa Creek system upstream of I-680. In order to compare the economic benefits of Alternatives 2B/d and 4/d on a consistent basis with remaining alternatives, economic benefits for Alternatives 2B/d and 4/d were developed assuming no future improvements are implemented on the Berryessa Creek system upstream of I-680.

5.2.2.1 Future Without Improvements Alternatives 2A/d, 2B/d, 4/d, and 5

Hydrologic inputs were developed for Alternatives 2A/d, 2B/d, 4/d, and 5 assuming that no future improvements planned by the SCVWD are constructed on the Berryessa Creek system upstream of I-680. The local and tributary inflow hydrographs for the future without improvements were taken from the future conditions 2003 HEC-HMS model corresponding to the values published in the NHC hydrology report (NHC 2003). The 2003 report does not include a number of future improvements planned by the SCVWD along the Berryessa Creek system. The 2003 hydrology was used to develop the Federal alternatives and to analyze the benefits of all alternatives. The 2006 NHC hydrology report (NHC 2006) reflects the future with improvements planned by the SCVWD. Since the addition of the SCVWD planned improvements would require a larger conveyance capacity and cost in the study area, the 2006 hydrology was used to develop the locally preferred alternatives and is discussed in the following section. Table 5-1 lists the peak discharges for each inflow hydrograph, HEC-RAS inflow station and HEC-HMS model nodes used to develop the inflow hydrographs. No changes were made to the hydrology for this study. The inflow hydrographs represent the flows entering the Berryessa Creek channel from I-680 downstream to just upstream of the confluence with Penitencia Creek. The unsteady HEC-RAS model allows the flows to escape the channel at the existing breakout locations covered in *Appendix B, Part II: Without-Project Floodplain Development.*

Alternative 2A/d was designed and economic benefits were derived based on the future without improvements hydrologic inputs. The economic benefits for Alternatives 2B/d and 4/d were derived based on the future without improvements hydrologic input with the alternative design based on the future with improvement hydrologic inputs as described in the following section.

Table 5-1 Discharges and Inflow Locations for Future Without Improvements

RAS Sta.	HMS Node	Description	Peak Discharge by Percent Chance Exceedance Event (cfs)							
			50%	20%	10%	4%	2%	1%	0.5%	0.2%
218+32	B13 RM 3.73	Subarea B12	269	382	461	692	811	928	1,073	1,227
174+48	B15 RM 2.96	Subarea B14	96	149	176	245	275	317	361	414
166+54	B17 RM 2.76	Piedmont Creek	244	387	450	715	821	858	900	900
144+67	B17a RM 2.58	Los Coches Creek	264	429	559	833	868	928	911	951
141+21	B19 RM 2.43	Calaveras Blvd Overflow	0	0	0	0	197	400	400	400
123+74	B21 RM 2.21	Tularcitos Creek	208	332	408	595	652	660	678	685
89+53	B23 RM 1.52	Berryessa Pump	107	150	150	150	150	150	150	150
74+86	B25 RM 1.22	Wrigley-Ford Pump	251	378	432	432	432	432	432	432
59+73	B27 RM 0.94	Calera Creek	180	292	367	521	669	869	1,099	1,261
56+27	B29 RM 0.77	Abbot Pump	583	851	1,041	1,330	1,436	1,568	1,676	1,710
51+66	B31 RM 0.14	Jurgens Pump	127	150	150	150	150	150	150	150
49+62	B 33 RM 0.00	Cal Circle Pump	22	30	34	42	48	56	63	71

Source: NHC (2003)

5.2.2.2 Future With Improvements – Alternatives 2B/d and 4/d

Hydrologic inputs were developed for Alternatives 2B/d and 4/d assuming that future improvements planned by the SCVWD upstream of I-680 are constructed on the Berryessa Creek system. The local and tributary inflow hydrographs were taken from the future conditions 2006 HEC-HMS model corresponding to the values published in the NHC hydrology report (NHC 2006). Since the planned SCVWD improvements require additional conveyance capacity in the study area, Alternatives 2B/d and 4/d are locally preferred alternatives. The 2006 hydrology was used to size the alternatives and the 2003 hydrology was used to analyze the resulting benefits. Table 5-2 lists the peak discharges for each inflow hydrograph, HEC-RAS inflow station and HEC-HMS model nodes used to develop the inflow hydrographs. No changes were made to the hydrology for this study. The discharge hydrographs represent the inflows to the Berryessa Creek channel from I-680 downstream to just upstream of the Penitencia Creek confluence. The unsteady HEC-RAS model allows the flows to escape the channel at the existing breakout locations covered in *Appendix B, Part II: Without-Project Floodplain Development*.

Alternatives 2B/d and 4/d were designed based on the future with improvement hydrologic input with the economic benefits for the alternatives based on the future without improvement hydrologic inputs described in the previous section.

Table 5-2 Discharges and Inflow Locations for Future With Improvements

RAS Sta.	HMS Node	Description	Peak Discharge by Percent Chance Exceedance Event (cfs)							
			50%	20%	10%	4%	2%	1%	0.5%	0.2%
218+32	B13 RM 3.73	Subarea B12	269	382	461	692	811	928	1,073	1,227
174+48	B15 RM 2.96	Subarea B14	96	149	176	245	275	317	361	414
166+54	B17 RM 2.76	Piedmont Creek	231	373	444	718	955	1,154	1,378	1,576
144+67	B17a RM 2.58	Los Coches Creek	263	427	556	803	1,015	1,297	1,626	1,898
141+21	B19 RM 2.43	Calaveras Blvd Overflow	0	0	0	0	0	0	0	0
123+74	B21 RM 2.21	Tularcitos Creek	187	294	361	527	653	826	974	1,146
89+53	B23 RM 1.52	Berryessa Pump	107	150	150	150	150	150	150	150
74+86	B25 RM 1.22	Wrigley-Ford Pump	251	378	432	432	432	432	432	432
59+73	B27 RM 0.94	Calera Creek	180	292	367	521	669	869	1,099	1,261
56+27	B29 RM 0.77	Abbot Pump	583	851	1,041	1,330	1,436	1,568	1,676	1,710
51+66	B31 RM 0.14	Jurgens Pump	127	150	150	150	150	150	150	150
49+62	B 33 RM 0.00	Cal Circle Pump	22	30	34	42	48	56	63	71

Note: Rows highlighted in gray represent location where the 2006 HEC-HMS modeling differs from the 2003 modeling as used for the existing conditions modeling described in Section 2.1.1.1. Source: NHC 2006

5.2.3 I-680 Culvert Outflow

Three different conditions were considered upstream of I-680. For the without-project and Alternative 2A/d conditions, the inflow at I-680 assumes that no future project is in place upstream of the interstate and the existing conditions prevail. Alternatives 2B/d and 4B/d assume that a bypass system, designed and built by the SCVWD, is in place above I-680. Alternative 5 assumes that the portion of the Authorized Project above I-680 is constructed at the same time as the portion below I-680. The following sections describe the development of the I-680 inflow hydrograph for use in the final array of alternatives.

5.2.3.1 No Bypass –Alternative 2A/d, 2B/d, and 4/d

The inflow hydrograph at I-680 for the No Bypass is the same as described in Section 2.2.1. Alternative 2A/d was designed and economic benefits were derived based on I-680 outflow hydrographs with no bypass upstream of I-680. The economic benefits for Alternatives 2B/d and 4/d were derived based on I-680 outflow hydrographs with no bypass upstream of I-680 with the alternatives designed based on I-680 outflow hydrographs with an upstream bypass in-place as described in the following section.

5.2.3.2 Upstream Bypass – Alternatives 2B/d and 4/d

Alternative 2B/d and Alternative 4/d were designed with different assumptions for the Berryessa Creek channel upstream of I-680 than those developed for the existing conditions modeling. SCVWD developed hydrologic and hydraulic modeling to analyze a proposed bypass culvert for Berryessa Creek upstream of I-680. The SCVWD bypass hydrology was used only to size the locally preferred alternatives 2/d and 4/d to ensure the alternatives were sized sufficiently to convey the resulting additional flow through study area. The resulting locally preferred alternatives were then analyzed using the Corps-approved 2003 hydrology. The bypass channel would begin at the upstream end of the Piedmont/Cropley Culvert and re-enter Berryessa Creek downstream of the Cropley Avenue Bridge with the bypass culvert alignment running underneath Cropley Avenue.

(a) Bypass Alternative Sizing Methodology

The hydraulic modeling of the bypass culvert was conducted using the Corps existing conditions HEC-RAS model with the baseline geometry used as the basis of the hydraulic analysis. The bypass culvert was modeled as a junction loop with the inlet junction of the bypass culvert located at the upstream end of the Piedmont/Cropley Bridge and the outlet junction located at the downstream end of the Cropley Bridge.

The hydrologic modeling of the bypass culvert was conducted using HEC-HMS model originally developed by Northwest Hydraulic Consultants (NHC 2003) in 2003 and updated in 2006 (NHC 2006). The future conditions basin configurations were used as the basis of the hydrologic analysis. The bypass culvert was added as a diversion card located downstream of node “B5 – Piedmont Road”, a junction card located below node “B11 – Morrill Road”, and a connecting routing reach.

The sizing of the bypass culvert and inlet was developed based on a targeted maximum flow of 400 cfs downstream of the Sweigert Creek confluence for the 0.01 chance exceedance event. The Sweigert Creek confluence is located about 1,000 feet downstream of the Piedmont/Cropley Culvert. The peak flow at Sweigert Creek is 308 cfs for the 0.01 chance exceedance event for Berryessa Creek. This flow results in a maximum release below the Piedmont/Cropley Culvert of 90 to 100 cfs to meet the target discharge of 400 cfs downstream Sweigert Creek.

The bypass culvert was sized using the HEC-RAS and HEC-HMS bypass models. First, the HEC-RAS model was run to develop the split flow rating curve based on a bypass culvert and inlet sizing. The split flow rating curve was then entered in the HEC-HMS model diversion card and the routing reach dimensions adjusted as needed. The HEC HMS model was then run and the discharge at the Sweigert Creek confluence was checked against the target discharge. The process was repeated iteratively until the target discharge downstream of Sweigert Creek was met.

(b) Bypass Alternative Design

The bypass alternative was developed using the methodology stated in the previous section by SCVWD. The bypass alternative was based on available data as of January 10, 2011 and was developed at a feasibility level for planning purposes only. The details of the assumed bypass structure will be fully developed by SCVWD during the design phase, and the resulting bypass rating curves may change.

The bypass culvert consists of 5,730 feet of 15-foot by 6-foot box culvert at a slope of 0.017 feet per feet (ft/ft). The invert of the bypass culvert inlet would be 2.5 feet vertically above the existing Piedmont/Cropley Culvert. The existing Piedmont/Cropley Culvert inlet would be modified to a 6.5-foot by 1.6-foot culvert from the existing 12-foot by 7-foot culvert. Figure 5-1 shows a conceptual layout of the bypass structure inlet. Table 5-3 lists the inflow, diverted bypass, and downstream outflow discharges for the bypass as described by SCVWD (2011a, 2011b).

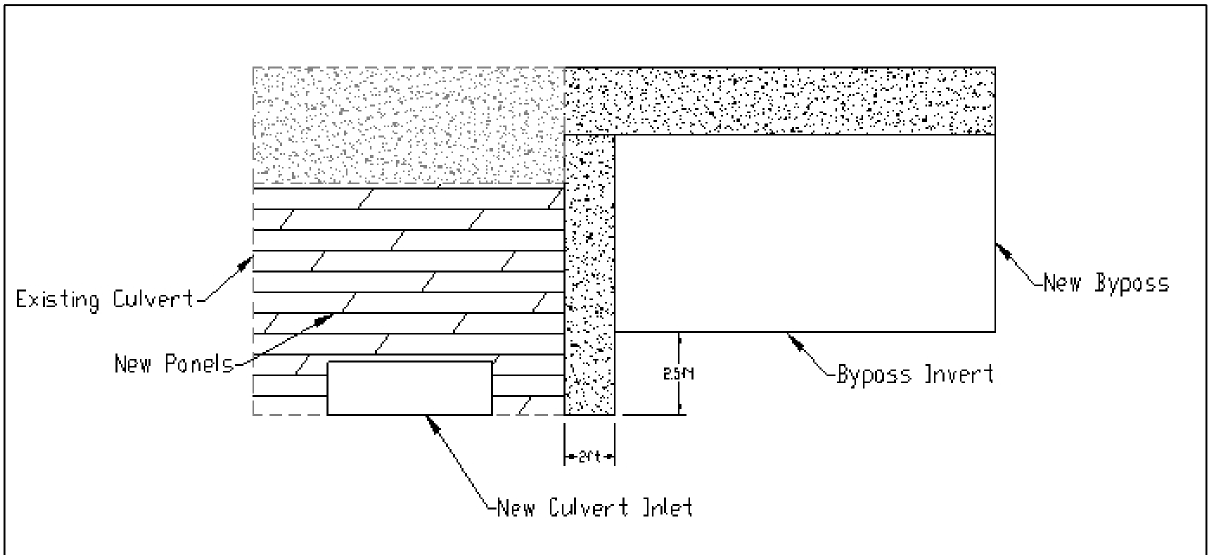

Figure 5-1 Conceptual Bypass Structure Inlet (Source: SCVWD 2011b)

Table 5-3 Inflow, Diverted, and Outflow Discharges at Bypass Structure

Percent Chance Exceedance Event	Berryessa Creek Inflow above Bypass Structure	Flow Diverted to Bypass Culvert	Berryessa Creek Outflow below Bypass Structure
	cfs	cfs	cfs
	0	0	0
	60	0	60
	100	17	83
50%	240	150	90
	340	245	95
20%	420	323	97
10%	560	458	102
4%	830	722	108
2%	1090	978	112
1%	1430	1310	120
0.5%	1820	1692	128
0.2%	2130	1994	136

Source: SCVWD 2011a, 2011b

(c) Bypass Alternative Results

The Berryessa Creek hydrographs at I-680 are used as the upstream input into the Lower Berryessa Creek HEC-RAS model for use in the development of the alternatives with upstream bypass in-place. The hydrographs used at I-680 were taken from node “B11 w Bypass” in the provided SCVWD Bypass HEC-HMS model (SCVWD 2011c). Table 5-4 lists the peak discharge, total volume, and time to peak for each of the flow events. Figure 5-2 shows the hydrographs at I-680 for the 50% to 0.2% chance exceedance events used. The outflow hydrographs from I-680 with the upstream bypass in-place was then used to design Alternatives 2B/d and 4/d. The economic benefits for Alternatives 2B/d and 4/d were then developed based on I-680 outflow hydrographs with no bypass upstream as described in the previous section.

Table 5-4 Peak Flow, Volume and Time to Peak for Bypass Alternative at I-680

Percent Chance Exceedance event	Peak Discharge (cfs)	Hydrograph Volume (ac-ft)	Time to Peak (hr)
50%	467	292.6	14.25
20%	719	437.9	14.25
10%	889	536.6	15.0
4%	1292	765.5	15.0
2%	1687	986.9	15.0
1%	2173	1350.8	15.0
0.5%	2742	1952.0	15.0
0.2%	3415	2387.8	14.0

Source: SCVWD 2011c

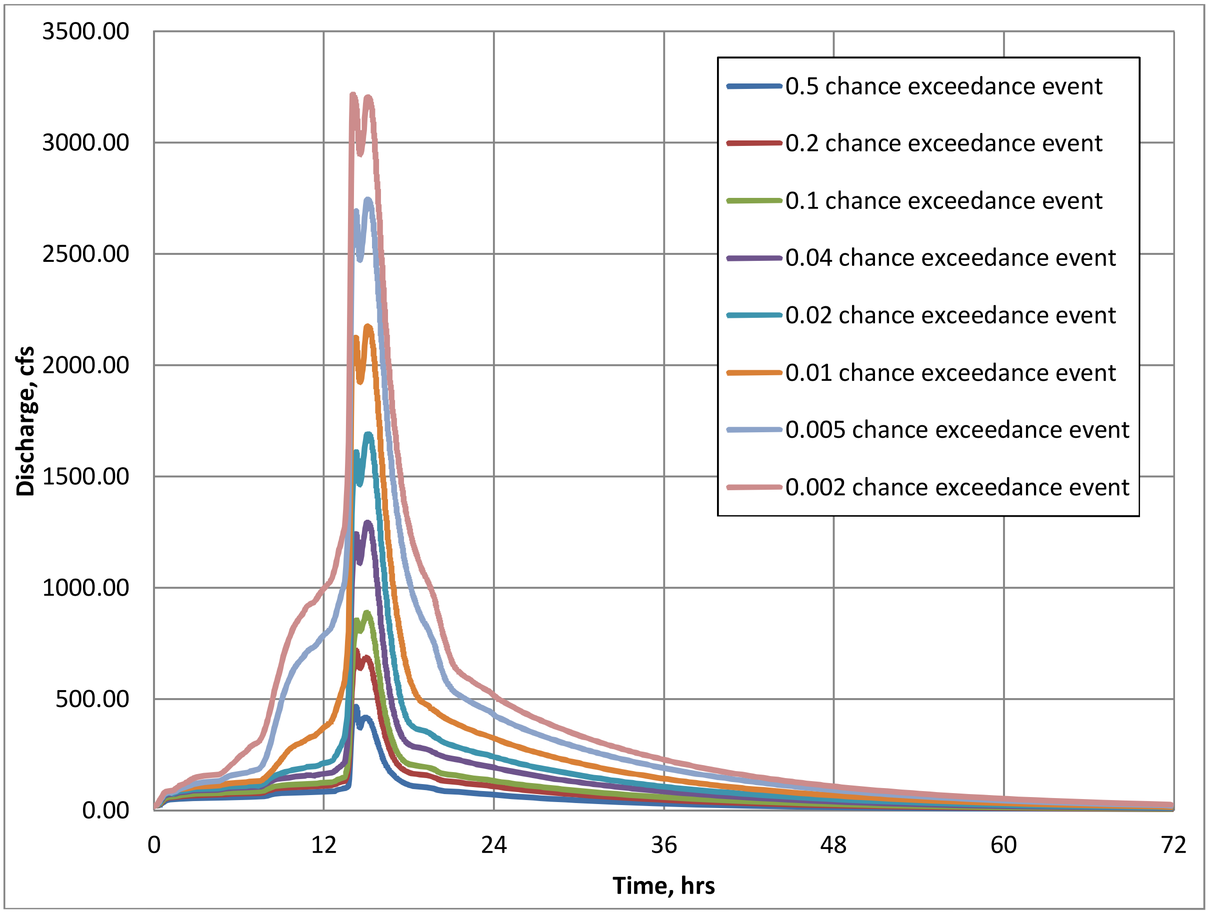

Figure 5-2 Hydrographs at I-680 for 50 to 0.2% Chance Exceedance Events (Source: SCVWD 2011c)

5.2.3.3 Authorized Project – Alternative 5

The I-680 culvert outflow hydrographs for Alternative 5 were developed from the Alternative 5 Upper Berryessa FLO-2D Model (see *Appendix B, Part II: Without-Project Floodplain Development*). Table 5-5 lists the peak discharges for each inflow hydrograph used in the Alternative 5 HEC-RAS model.

Table 5-5 Peak Flow, Volume and Time to Peak for Authorized Project at I-680

Percent Chance Exceedance event	Peak Discharge (cfs)	Hydrograph Volume (ac-ft)	Time to Peak (hr)
50%	482	433.9	14.5
20%	677	679.0	14.5
10%	849	792.6	15.5
4%	1208	941.9	15.5
2%	1526	1091.1	15.5
1%	1988	1339.5	15.5
0.5%	2310	1817.6	15.5
0.2%	2358	2128.2	15.75

5.2.4 Geometry

The geometries for the four alternatives were taken from the geometries developed for the preliminary array of alternatives as described in Chapter 4. The geometry file for each alternative was then modified to eliminate the reach and all associated cross sections above cross section 25471. This cross section represents the outlet of the I-680 culvert and is the upstream end of the revised GRR HEC-RAS model.

Project features, including the hydraulic structure capacities and top of bank/levee elevations, for Alternative 2A from the preliminary array of alternatives were revised to pass the 1% chance exceedance event. The minimum cross section considered was a cross section with a 10-foot bottom width and an in-channel maintenance road. From approximately downstream of Montague Avenue to Yosemite Avenue the minimum cross section was used resulting in a channel that is able to convey more than the 1% chance exceedance event. To reduce the channel cross section to a point where the channel would just convey the 1% chance exceedance event in this section would result in a channel section that does not fulfill the design criteria for Alternative 2A. Therefore Alternative 2A consists of three sections:

- Upstream of Montague Avenue – Designed to pass the 1% chance exceedance event
- Montague Avenue to Yosemite Avenue – Designed using the minimum channel cross section
- Downstream of Yosemite Avenue – Designed to pass the 1% chance exceedance event

A full description of the all of the alternatives in the final array of alternatives is included in *Appendix B, Part IV: Design and Cost of Alternatives*

Project features including the hydraulic structure capacities and top of bank/levee elevations for Alternatives 2B and 4 from the preliminary array of alternatives were revised to meet the requirements for FEMA Certification using risk and uncertainty principles per Engineering Circular 1110-2-6067, *Certification of Levee Systems for the National Flood Insurance Program* (USACE 2008a).

For Alternative 5 no further changes were made to the channel cross sections, bridges, or culverts downstream of station 25471.

The Berryessa Creek reach upstream of I-680 was removed from each alternative and the hydrologic inputs were developed to allow for unsteady runs to be made. The resulting alternatives are designated as 2A/d, 2B/d, and 4/d to indicate that they only include project features for Berryessa Creek downstream of I-680. Alternative 5 remains the same and includes all project elements upstream of I-680.

Alternatives 2B/d and 4/d were divided into five reaches with representative index cross section assigned to each reach. The Corps HEC-FDA program version 1.2.5a was then used to determine the CNP. The hydraulic and hydrologic data developed for the GRR were used as inputs, along with the top of levee elevations to determine the CNP for each reach. Each reach was analyzed to determine if a minimum CNP of 90% for the 1% chance exceedance event was achieved for entrenched channels. Based on the CNP results the alternatives were refined as needed and the process repeated until the desired minimum CNP of 90% was reached or exceeded (USACE 2008a). The following sections describe the development of the project performance for Alternatives 2B/d and 4/d and the results are presented in Section 5.3.6.

5.3 Project Performance

The conditional CNP for the alternatives was used to quantify the project performance for the study alternatives and ensure that each alternative met the minimum project performance criteria specified for the alternative. Each alternative was developed in order to meet a minimum CNP of 90% for the 1% chance exceedance event. The CNP is an index of the likelihood that a specified target stage will not be exceeded, given the occurrence of a hydrometeorological event (USACE 1994). The project performance was developed for this study using USACE' Flood Damage Assessment software, HEC-FDA version 1.2.5a. HEC-FDA requires the following inputs to calculate the CNP:

- Stage-Frequency, Stage-Discharge and Discharge-Probability curves to represent the Water Surface Profile
- Stage-Discharge uncertainty
- Discharge-Probability uncertainty
- Economic Input Data
- Target stage

The following sections describe the inputs into HEC-FDA and the subsequent results.

5.3.1 Water Surface Profiles

The stage-discharge and discharge probability relationships were developed for five reaches along Berryessa Creek downstream of I-680 and six reaches upstream of I-680 for the without-project conditions for each alternative[8]. Table 5-6 lists the index locations and the bounding HEC-RAS cross section stations for each index location. The stage-discharge and discharge-probability relationships for the index sections were developed using the Revised Lower HEC-RAS model as described in Chapter 2 for the reaches downstream of I-680 and from the Upper Berryessa FLO-2D model as described in *Appendix B, Part II: Without-Project Floodplain Development for the reaches upstream of I-680*. Table 5-7 lists the stage-discharge relationships for each index location for the without-project, Alternative 2A/d, and Alternative 5. The stage-discharge and discharge-probability relationships listed assumes future without improvement upstream of I-680 for the without-project, and Alternative 2A/d and for Alternative 5 assume that the upstream components of the alternative are in-place upstream of I-680. Table 5-8 lists the stage-discharge and discharge-probability relationships for Alternative 2B/d and Alternative 4/d for both future without- and with-improvements upstream of I-680. The future with-improvements upstream of I-680 (SCVWD bypass structure in-place and miscellaneous other improvements, see Section 5.2 for details) stage-discharge and discharge probability relationships were used during the design the alternatives. The future without-improvements upstream of I-680 stage-discharge and discharge probability relationships were used to determine the economic benefits of the alternatives.

[8] The hydraulic reaches discussed in this appendix refer to the hydraulic reaches specified in the scope of work to ensure hydraulic performance goals were met. The Economic Appendix discusses the results of the economic analysis on economic reaches developed independently of the hydraulic reaches, based on economic criteria. The reaches referenced in this and the economic appendix are independent and are not meant to correlate between appendices.

Table 5-6 Stage-Discharge Uncertainty Reaches

Reach	HEC-RAS Station/ FLO-2D Gird Location			Watershed Area at Index location (sq mi)
	Downstream	Index	Upstream	
US Extent to Old Piedmont Road	3106	3107	3142	4.4
Old Piedmont Road to Piedmont-Cropley	3038	3039	3075	4.9
Piedmont-Cropley to Drop Structure US of Morrill Ave.	1566	2423	2967	5.8
Drop Structure US of Morrill Ave. to Morrill Ave.	1279	1375	1471	7.7
Morrill Ave. to Cropley Ave.	890	986	1230	7.8
Cropley Ave. to I-680	43	418	840	7.9
I-680 to Montague Blvd	25575	22274	21738	8.83
Montague to UPRR Trestle	21738	21601	21247	8.93
UPRR Trestle to UPRR Triple Box	21274	20131	19333	9.02
UPRR Triple Box to Ames Ave	19333	19158	18843	9.09
Ames Ave to Calaveras Blvd	18843	16924	13803	10.52

Table 5-7 Stage-Discharge and Discharge-Probability Relationship for Lower Berryessa Creek Index locations (Without-Project, Alt 2A/d and Alt 5)

Reach	Index Grid Cell/ Cross Section	Percent Chance Exceedance Event	Without-Project Conditions		Alt 2A/d		Alt 5	
			Stage	Q	Stage	Q	Stage	Q
			ft	Cfs	ft	cfs	ft	Cfs
US Extent to Old Piedmont Road	3107[1]	50%	213.70	240	Same as Without-Project Condition		211.19	240
		20%	214.28	420			212.66	420
		10%	215.12	560			213.80	560
		4%	216.88	830			215.24	830
		2%	219.26	1090			216.70	1090
		1%	220.15	1430			218.51	1421
		0.5%	221.39	1820			219.38	1854
		0.2%	222.31	2130			223.14	2130
Old Piedmont Road to Piedmont-Cropley	3039[1]	50%	190.63	255	Same as Without-Project Condition		196.48	280
		20%	191.64	456			197.30	480
		10%	192.48	614			197.86	642
		4%	193.91	880			198.57	911
		2%	195.66	1147			199.15	1219
		1%	197.27	1468			199.16	1439
		0.5%	197.97	1721			200.87	1880
		0.2%	198.50	1924			202.17	2037
Piedmont-Cropley to Drop Structure US of Morrill Ave.	2423[1]	50%	145.40	265	Same as Without-Project Condition		142.38	260
		20%	146.09	444			146.10	443
		10%	146.34	598			146.36	594
		4%	146.70	860			146.78	854
		2%	146.89	1047			147.02	1109
		1%	146.91	1052			147.27	1433
		0.5%	146.93	1098			147.42	1635
		0.2%	146.94	1114			147.44	1664
Drop Structure US of Morrill Ave. to Morrill Ave.	1375[1]	50%	109.49	306	Same as Without-Project Condition		102.86	378
		20%	110.46	511			103.91	747
		10%	111.16	671			104.34	747
		4%	112.32	897			105.91	951
		2%	113.02	1033			107.47	1284

Reach	Index Grid Cell/ Cross Section	Percent Chance Exceedance Event	Without-Project Conditions		Alt 2A/d		Alt 5	
			Stage	Q	Stage	Q	Stage	Q
			ft	Cfs	ft	cfs	ft	Cfs
		1%	113.44	1133			110.38	1605
		0.5%	113.75	1313			112.49	1876
		0.2%	114.22	1436			112.77	1904
Morrill Ave. to Cropley Ave.	986[1]	50%	99.26	477	Same as Without-Project Condition		96.31	485
		20%	100.23	694			97.16	685
		10%	100.89	852			97.80	863
		4%	103.12	1171			98.94	1211
		2%	104.48	1427			100.79	1541
		1%	104.69	1589			102.51	1999
		0.5%	104.81	1667			104.35	2368
		0.2%	105.03	1790			104.57	2433
Cropley Ave. to I-680	418[1]	50%	87.47	474	Same as Without-Project Condition		84.86	484
		20%	88.31	690			85.60	685
		10%	88.82	845			86.08	853
		4%	89.73	1148			87.12	1220
		2%	90.46	1408			87.83	1538
		1%	90.79	1547			88.86	1996
		0.5%	90.95	1612			89.43	2323
		0.2%	91.23	1724			89.65	2360
I-680 to Montague Blvd	22274[2]	50%	61.63	483	58.20	485	57.67	479
		20%	62.59	692	59.23	695	59.28	675
		10%	63.58	923	60.11	926	60.06	755
		4%	64.50	964	61.07	995	62.06	980
		2%	64.71	1100	61.59	1079	63.12	1148
		1%	64.86	1143	64.15	1184	64.62	1393
		0.5%	65.01	1200	65.28	1425	65.32	1716
		0.2%	65.07	1207	65.48	1452	65.50	1924
Montague to UPRR Trestle	21601[2]	50%	58.58	630	55.84	629	55.76	638
		20%	59.83	962	56.98	961	56.74	947
		10%	60.76	1234	57.85	1246	57.22	1107
		4%	61.57	1442	58.80	1583	58.47	1563

Reach	Index Grid Cell/ Cross Section	Percent Chance Exceedance Event	Without-Project Conditions		Alt 2A/d		Alt 5	
			Stage	Q	Stage	Q	Stage	Q
			ft	Cfs	ft	cfs	ft	Cfs
		2%	61.93	1483	59.31	1771	59.19	1831
		1%	62.17	1505	60.06	2057	60.13	2244
		0.5%	62.38	1554	61.11	2437	60.72	2518
		0.2%	62.51	1592	61.28	2510	60.79	2567
UPRR Trestle to UPRR Triple Box	20131[2]	50%	52.04	629	49.62	629	50.10	637
		20%	53.32	960	50.74	959	50.93	947
		10%	54.14	1231	51.55	1241	51.30	1106
		4%	54.74	1441	52.42	1573	52.31	1561
		2%	54.87	1482	52.89	1763	52.87	1828
		1%	54.93	1505	53.56	2045	53.74	2238
		0.5%	55.07	1553	54.70	2409	55.11	2525
		0.2%	55.17	1589	54.88	2501	55.25	2587
UPRR Triple Box to Ames Ave	19158[2]	50%	47.79	628	46.65	627	45.77	636
		20%	49.10	959	47.86	957	46.91	946
		10%	50.01	1229	48.73	1238	47.41	1105
		4%	50.65	1440	49.64	1569	48.65	1559
		2%	50.82	1481	50.11	1761	49.30	1826
		1%	50.90	1504	50.74	2028	50.33	2231
		0.5%	51.04	1553	52.51	2406	53.24	2525
		0.2%	51.14	1589	52.63	2499	53.37	2584
Ames Ave to Calaveras Blvd	16924[2]	50%	36.80	676	35.01	676	34.14	685
		20%	37.76	923	35.94	1019	34.94	1017
		10%	37.86	1300	36.59	1306	35.32	1193
		4%	38.13	1520	37.53	1690	36.29	1686
		2%	38.21	1543	37.86	1896	36.78	1963
		1%	38.31	1601	38.20	2187	37.35	2339
		0.5%	38.33	1683	38.56	2450	37.75	2621
		0.2%	38.35	1685	38.73	2477	37.99	2819

Note: 1. FLO-2D Grid Cell
2. HEC-RAS Cross Section

Table 5-8 Stage-Discharge and Discharge-Probability Relationship for Lower Berryessa Creek Index locations or Future With and Without Improvements Upstream of I-680 (Alt 2B/d and Alt 4/d)

Reach	Index Section	Percent Chance Exceedance Event	Future No Improvements Upstream of I-680				Future With Improvements Upstream of I-680			
			Alt 2B/d		Alt 4/d		Alt 2B/d		Alt 4/d	
			Stage	Q	Stage	Q	Stage	Q	Stage	Q
			ft	cfs	ft	cfs	ft	cfs	ft	cfs
US Extent to I-680			Same as Without-Project Condition listed in Table 5-13				Hydraulic Analysis not Conducted for Locally Developed Future Upstream Improvements[1]			
I-680 to Montague Blvd	22274	50%	58.97	485	58.42	486	58.84	459	58.36	460
		20%	59.86	695	58.94	697	59.88	705	58.97	707
		10%	60.46	953	59.47	929	60.31	885	59.31	842
		4%	60.86	1143	59.98	1002	61.25	1228	60.41	1230
		2%	61.39	1394	60.36	1095	62.13	1593	61.42	1594
		1%	61.70	1542	61.00	1296	63.39	2110	63.02	2112
		0.5%	62.49	1538	61.97	1600	64.52	2660	64.52	2666
		0.2%	62.95	1607	62.55	1612	65.45	3192	66.19	3178
Montague to UPRR Trestle	21601	50%	55.76	628	55.85	628	55.87	660	55.92	658
		20%	56.89	959	56.66	959	56.97	984	56.72	984
		10%	57.72	1245	57.36	1246	57.52	1173	57.19	1175
		4%	58.57	1579	58.19	1578	59.01	1765	58.68	1761
		2%	59.02	1767	58.68	1761	60.02	2229	59.87	2230
		1%	59.65	2054	59.40	2051	61.41	2939	61.70	2935
		0.5%	60.53	2480	60.53	2477	62.60	3604	63.33	3599
		0.2%	61.01	2724	61.16	2720	63.64	3901	64.58	4135
UPRR Trestle to UPRR Triple Box	20131	50%	49.53	628	49.53	627	49.62	654	49.59	653
		20%	50.57	958	50.22	958	50.62	982	50.26	982
		10%	51.17	1243	50.74	1243	51.02	1171	50.62	1173
		4%	51.81	1577	51.31	1574	52.14	1764	51.62	1758
		2%	52.15	1766	51.62	1758	52.90	2228	52.39	2226
		1%	52.63	2052	52.10	2049	53.84	2937	53.50	2927
		0.5%	53.25	2478	52.83	2469	54.63	3602	54.69	3590
		0.2%	53.57	2722	53.14	2660	55.35	4213	55.66	4121
UPRR	19158	50%	45.80	627	46.26	625	45.88	652	46.32	649

Reach	Index Section	Percent Chance Exceedance Event	Future No Improvements Upstream of I-680				Future With Improvements Upstream of I-680			
			Alt 2B/d		Alt 4/d		Alt 2B/d		Alt 4/d	
			Stage	Q	Stage	Q	Stage	Q	Stage	Q
			ft	cfs	ft	cfs	ft	cfs	ft	cfs
Triple Box to Ames Ave		20%	46.92	957	46.97	956	46.97	980	47.02	979
		10%	47.55	1239	47.49	1239	47.42	1167	47.37	1169
		4%	48.11	1575	48.03	1569	48.39	1763	48.33	1756
		2%	48.39	1764	48.33	1755	49.00	2226	49.00	2222
		1%	48.78	2051	48.75	2047	49.83	2935	49.91	2925
		0.5%	49.31	2476	49.32	2466	50.52	3601	50.69	3589
		0.2%	49.59	2721	49.65	2716	51.72	4203	52.54	4105
Ames Ave to Calaveras Blvd	16924	50%	34.84	676	36.52	674	34.92	722	36.62	716
		20%	35.84	1019	37.37	1016	35.90	1050	37.44	1045
		10%	36.53	1311	37.97	1307	36.42	1246	37.87	1242
		4%	37.45	1695	38.91	1685	37.68	1882	39.18	1869
		2%	37.83	1902	39.36	1886	38.43	2349	40.10	2338
		1%	38.19	2203	39.83	2192	39.35	3097	41.30	3082
		0.5%	38.65	2652	40.43	2626	40.15	3778	42.38	3761
		0.2%	38.93	2937	40.80	2926	41.02	4401	43.22	4271

Note: 1. Locally Developed Future Upstream Improvements are described in Section 5.2.2

5.3.2 Stage-Discharge Uncertainty

The stage-discharge uncertainty accounts for the uncertainty associated with the factors affecting the stage-discharge relationship. These factors can include, but are not limited to, the following:

- bed forms
- water temperature
- debris or other obstructions
- unsteady flow effects
- variation in hydraulic roughness with season, sediment transport, channel scour, or deposition
- changes in channel shape during or as a result of flood events

The procedures specified in EM 1110-2-1619 Risk Based Analysis for Flood Damage Reduction Studies (USACE 1996) were used to develop the stage-discharge uncertainties. In order to develop the stage-discharge uncertainty, two items were calculated. First, the natural uncertainty was developed using the procedure listed in Section 5-4 of EM 1110-2-1619.

Second, the modeling stage-discharge uncertainty from computed water surface elevation (WSEL) profiles was developed using the procedure listed in Section 5-7 of EM 1110-2-1619. The natural and modeling uncertainties are then combined to develop the stage-discharge uncertainty for the index location. The natural stage-discharge uncertainty for ungauged streams correlates stage uncertainty to measurable stream parameters. The equation for developing the stage-discharge uncertainty for ungauged streams is stated in EM 1110-2-1619 Equation 5-5 (COE, 1996) and is as follows:

$$S_{Natural} = [0.07208+0.04936*I_{Bed} - (2.2626*10^{-7})*A_{Basin}+0.02164 * H_{Range}+ (1.4194*10^{-5}) * Q_{100}]^2$$

Where:

$S_{Natural}$ = Standard deviation of natural uncertainty for ungauged stream, meters
I_{Bed} = Stream identifier based on size of bed material based on EM 1110-2-1619 Table 5-1 (COE 1996), dimensionless
A_{Basin} = Basin area at index location, square kilometers
H_{Range} = Maximum expected stage range, in meters
Q_{100} = 1% chance exceedance discharge at index location, cubic meters per second

Each variable was determined at each index location. I_{Bed} was assigned as sands due to the potential of erosion and deposition in the earthen reaches. H_{Range} was determined to be the channel depth at each location since flows in Berryessa Creek can range from no flow to bankfull. Finally, A_{Basin} and Q_{100} were determined from the available HEC-HMS modeling data. Table 5-9 lists the resulting natural uncertainty and related inputs for each of the index locations along Lower Berryessa Creek for each alternative. The stage-discharge uncertainty equation was developed in metric units. For the purposes of the GRR, the stage-discharge uncertainty results were calculated using the metric input values with the final results converted from meters to feet and presented in the table.

Table 5-9 Natural Uncertainty for Lower Berryessa Creek Index Locations

Reach	Index Section	I_{Bed}	A_{Basin} sq mi	H_{Range} Ft	Q_{100} Cfs	$S_{Natural}$ Ft
Without-Project Conditions						
US Extent to Old Piedmont Road	3107	4	4.4	8.5	1,430	0.35
Old Piedmont Road to Piedmont-Cropley	3039	4	4.9	13.0	1,467	0.42
Piedmont-Cropley to Drop Structure US of Morrill Ave.	2423	4	5.8	8.6	969	0.35
Drop Structure US of Morrill Ave. to Morrill Ave.	1375	3	7.7	9.4	1,133	0.36
Morrill Ave. to Cropley Ave.	986	4	7.8	8.6	1,589	0.35
Cropley Ave. to I-680	418	0	7.9	9.5	1,547	0.06
I-680 to Montague Blvd	22274	4	8.8	7.1	1,143	0.33
Montague to UPRR Trestle	21601	4	8.9	11.8	1,505	0.40
UPRR Trestle to UPRR Triple Box	20131	4	9.0	12.0	1,505	0.40
UPRR Triple Box to Ames Ave	19158	4	9.1	12.3	1,505	0.40
Ames Ave to Calaveras Blvd	16924	4	10.5	9.0	1,601	0.36
Alternative 2A/d						
Alternative 2A/d Natural Uncertainty for Reaches Upstream of I-680 same as Existing Conditions						
I-680 to Montague Blvd	22274	4	8.8	10.8	1,132	0.38
Montague to UPRR Trestle	21601	4	8.9	11.0	2,057	0.39
UPRR Trestle to UPRR Triple Box	20131	4	9.0	12.0	2,044	0.40
UPRR Triple Box to Ames Ave	19158	4	9.1	11.0	2,028	0.39
Ames Ave to Calaveras Blvd	16924	4	10.5	7.0	2,187	0.33
Alterative 2B/d						
Alternative 2B/d Natural Uncertainty for Reaches Upstream of I-680 same as Existing Conditions						
I-680 to Montague Blvd	22274	4	8.8	11.3	2,110	0.39
Montague to UPRR Trestle	21601	4	8.9	12.6	2,939	0.41
UPRR Trestle to UPRR Triple Box	20131	4	9.0	12.0	2,936	0.40
UPRR Triple Box to Ames Ave	19158	4	9.1	11.0	2,935	0.39
Ames Ave to Calaveras Blvd	16924	4	10.5	8.9	3,097	0.36
Alternative 4B/d						
Alternative 4/d Natural Uncertainty for Reaches Upstream of I-680 same as Existing Conditions						
I-680 to Montague Blvd	22274	4	8.8	11.1	2,112	0.39
Montague to UPRR Trestle	21601	4	8.9	13.3	2,935	0.42
UPRR Trestle to UPRR Triple Box	20131	4	9.0	12.0	2,925	0.40
UPRR Triple Box to Ames Ave	19158	4	9.1	11.3	2,925	0.39

Reach	Index Section	I_{Bed}	A_{Basin} sq mi	H_{Range} Ft	Q_{100} Cfs	$S_{Natural}$ Ft
Ames Ave to Calaveras Blvd	16924	4	10.5	11.9	3,082	0.40
Alternative 5						
US Extent to Old Piedmont Road	3107	0	4.4	12.5	1421	0.08
Old Piedmont Road to Piedmont-Cropley	3039	0	4.9	7.9	1439	0.05
Piedmont-Cropley to Drop Structure US of Morrill Ave.	2423	4	5.8	8.2	1433	0.34
Drop Structure US of Morrill Ave. to Morrill Ave.	1375	3	7.7	10.4	1604	0.28
Morrill Ave. to Cropley Ave.	986	0	7.8	11.2	1999	0.07
Cropley Ave. to I-680	418	0	7.9	10.0	1996	0.06
I-680 to Montague Blvd	22274	0	8.8	11.4	1393	0.07
Montague to UPRR Trestle	21601	0	8.9	11.4	2244	0.07
UPRR Trestle to UPRR Triple Box	20131	0	9.0	10.9	2238	0.07
UPRR Triple Box to Ames Ave	19158	0	9.1	10	2231	0.06
Ames Ave to Calaveras Blvd	16924	0	10.5	9.4	2338	0.06

The modeling stage-discharge uncertainty was developed using without-project and alternative geometries in the Revised Lower HEC-RAS model for the Berryessa Creek reaches downstream of I-680. The without-project and Alternative 5 Upper Berryessa FLO-2D models were used to develop the modeling stage-discharge uncertainty for the reaches upstream of I-680. The modeling stage-discharge uncertainty is defined as the standard deviation ($S_{Computed}$), which is defined as one-half of the difference between the mean and the upper limit WSEL profiles for each reach. The mean models were based on the Revised Lower HEC-RAS model geometries and Upper Berryessa FLO-2D models. The upper limit models were developed by increasing the Manning's n-values by 20% and adding sediment and pier debris loading.

The sediment loading was based on the average annual volume of sediment removed by SCVWD personel from each of five maintenance zones as described in *Appendix B, Part III: Geomorphic and Sediment Transport Assessment*. The average annual sediment removal volumes were reported for three maintenance reaches downstream of I-680 and two upstream of I-680. Based on the observations of David Adams of the SCVWD, sediment removed in the maintenance reaches upstream of Calaveras Boulevard is approximately uniformly distributed within each channel reach (rather than concentrated at bridge locations). The sediment removal volume for the final maintenance reach downstream of Calaveras was observed to be approximately 90% removed between Calaveras Boulevard and North Able Street with the remaining 10% removed between North Abel Street and the Penitencia Creek confluence. Based on the SCVWD maintenance observations, the average annual sediment removal was distributed over the following five zones with approximately uniform distribution within each zone:

- Zone 1 – Piedmont Sediment Basin – 527 cy
- Zone 2 – Sierra Creek to Cropley Avenue – 525 cy
- Zone 3- I-680 to Montague Boulevard – 440 cy
- Zone 4 - Montague Boulevard to Calaveras Boulevard – 230 cy
- Zone 5a - Calaveras Boulevard to North Abel Street – 4630 cy (90% of 5136 cy)
- Zone 5b - North Abel Street to Penitencia Creek Confluence – 514 cy (10% of 5136 cy)

The sediment deposition volume for each maintenance reach was uniformly distributed using the fixed sediment elevation tool in HEC-RAS and manually adjusting the cross sections in FLO-2D. Table 5-10 lists the average annual sediment deposition volume for each maintenance zone and resulting sediment deposition depth used in the upper limit model. For the reaches upstream of I-680 with no sediment maintenance records available, the following assumptions were made for sediment deposition:

- Upstream of Old Piedmont Road to Piedmont-Cropley Culvert Without-Project - assumes 0.25 feet of uniform deposition over reach
- Upstream of Old Piedmont Road to Piedmont-Cropley Culvert Alternative 5 - assumes 0.25 feet of uniform deposition over reach with the upstream sediment basin full
- Greenbelt Reach – assumes no sediment deposition
- Cropley Avenue to I-680 – assumes same deposition as Zone 2

Floating pier debris of 3 feet wide by 3 feet tall was added to the model at the Montague Blvd and Calaveras Blvd bridge piers based on pier debris removal observations provided by David Adams of the SCVWD. The same floating debris was added to the Morrill Avenue and Cropley Avenue bridges in the FLO-2D models. For the without-project FLO-2D model, the Piedmont-Cropley culvert was assumed to be in the same condition as it exists today, with sediment deposition in the culvert reducing capacity.

Table 5-10 Upper Limit Sediment Deposition Depths

	Maintenance Zone					
	Zone 1	**Zone 2**	**Zone 3**	**Zone 4**	**Zone 5a**	**Zone 5b**
Length (ft)	120	1194	3,800	8,230	5,900	1,790
Average Annual Sediment Deposition (cy)	527	525	440	230	4,630	514
Without-Project Conditions						
Ave. Bottom Width (ft)	32	25	10	15	76	111
Sediment Deposition Depth (ft)	0.18	0.46	0.31	0.05	0.28	0.05
Alternative 2A/d						
Ave. Bottom Width (ft)	Same as Without-Project Condition		14	12	73	111
Sediment Deposition Depth (ft)			0.13	0.06	0.29	0.07
Alternative 2B/d						
Ave. Bottom Width (ft)	Same as Without-Project Condition		24	38	73	111
Sediment Deposition Depth (ft)			0.13	0.02	0.29	0.07
Alternative 4/d						
Ave. Bottom Width (ft)	Same as Without-Project Condition		12	13	76	111
Sediment Deposition Depth (ft)			0.26	0.06	0.28	0.07
Alternative 5						
Ave. Bottom Width (ft)	32[1]	25[1]	20	15	73	111
Sediment Deposition Depth (ft)	0.18[1]	0.46[1]	0.31	0.05	0.29	0.05

Note: 1. Alternative 5 retains the existing channel configuration in Zones 1 and 2 resulting in the same sediment deposition depths.

The mean and upper limit geometries were run using the 1% chance exceedance event inflow file. The difference in the resulting WSEL profiles was then calculated for each cross section. The average difference for each reach was computed as linearly-weighted average of the cross section differences in each uncertainty reach. The modeling uncertainty standard deviation ($S_{Computed}$) was computed as one-half of the reach average difference. Table 5-11 lists the modeling stage-discharge uncertainty for each index location.

The total stage-discharge uncertainty (S_{Total}) is a combination of the natural and modeling uncertainties and is defined in EM 1110-2-1619 (COE 1996) as follows:

$$S_{Total} = (S_{Natural}^2 + S_{Computed}^2)^{0.5}$$

Where:

S_{Total} = standard deviation of the total uncertainty
$S_{Natural}$ = natural uncertainty
$S_{Computed}$ = modeling uncertainty

Table 5-11 lists the Natural, Computed, and Total stage-discharge uncertainty for each index location.

Table 5-11 Total Stage-Discharge Uncertainty for Lower Berryessa Creek Index Locations

Reach	**Index Section**	$S_{Natural}$ **Ft**	$S_{Computed}$ **Ft**	S_{Total} **Ft**	S_{Total} **Adopted Ft**
Without-Project Conditions					
US Extent to Old Piedmont Road	3107	0.35	0.15	0.38	0.9
Old Piedmont Road to Piedmont-Cropley	3039	0.42	0.17	0.45	0.9
Piedmont-Cropley to Drop Structure US of Morrill Ave.	2423	0.35	0.10	0.36	0.9
Drop Structure US of Morrill Ave. to Morrill Ave.	1375	0.36	0.29	0.46	0.9
Morrill Ave. to Cropley Ave.	986	0.35	0.05	0.35	0.9
Cropley Ave. to I-680	418	0.06	0.13	0.15	0.9
I-680 to Montague Blvd	22274	0.33	0.18	0.37	0.9
Montague to UPRR Trestle	21601	0.40	0.07	0.40	0.9
UPRR Trestle to UPRR Triple Box	20131	0.40	0.15	0.43	0.9
UPRR Triple Box to Ames Ave	19158	0.40	0.09	0.41	0.9
Ames Ave to Calaveras Blvd	16924	0.36	0.08	0.37	0.9
Alternative 2A/d					
Alternative 2A/d Natural Uncertainty for Reaches Upstream of I-680 same as Existing Conditions					
I-680 to Montague Blvd	22274	0.38	0.38	0.54	0.9
Montague to UPRR Trestle	21601	0.39	0.61	0.72	0.9
UPRR Trestle to UPRR Triple Box	20131	0.40	0.61	0.73	0.9
UPRR Triple Box to Ames Ave	19158	0.39	0.88	0.96	0.96
Ames Ave to Calaveras Blvd	16924	0.33	0.51	0.61	0.9
Alternative 2B/d					
Alternative 2B/d Natural Uncertainty for Reaches Upstream of I-680 same as Existing Conditions					

Reach	Index Section	$S_{Natural}$ Ft	$S_{Computed}$ Ft	S_{Total} Ft	S_{Total} Adopted Ft
I-680 to Montague Blvd	22274	0.39	0.31	0.50	0.9
Montague to UPRR Trestle	21601	0.41	0.37	0.56	0.9
UPRR Trestle to UPRR Triple Box	20131	0.40	0.34	0.53	0.9
UPRR Triple Box to Ames Ave	19158	0.39	0.30	0.49	0.9
Ames Ave to Calaveras Blvd	16924	0.36	0.31	0.48	0.9
Alternative 4/d					
Alternative 4/d Natural Uncertainty for Reaches Upstream of I-680 same as Existing Conditions					
I-680 to Montague Blvd	22274	0.39	0.28	0.48	0.9
Montague to UPRR Trestle	21601	0.42	0.05	0.42	0.9
UPRR Trestle to UPRR Triple Box	20131	0.40	0.32	0.51	0.9
UPRR Triple Box to Ames Ave	19158	0.39	0.25	0.47	0.9
Ames Ave to Calaveras Blvd	16924	0.40	0.33	0.52	0.9
Alternative 5					
US Extent to Old Piedmont Road	3107	0.08	0.21	0.22	0.9
Old Piedmont Road to Piedmont-Cropley	3039	0.05	0.14	0.15	0.9
Piedmont-Cropley to Drop Structure US of Morrill Ave.	2423	0.34	0.14	0.37	0.9
Drop Structure US of Morrill Ave. to Morrill Ave.	1375	0.28	0.05	0.28	0.9
Morrill Ave. to Cropley Ave.	986	0.07	0.21	0.23	0.9
Cropley Ave. to I-680	418	0.06	0.31	0.31	0.9
I-680 to Montague Blvd	22274	0.07	0.23	0.24	0.9
Montague to UPRR Trestle	21601	0.07	0.10	0.12	0.9
UPRR Trestle to UPRR Triple Box	20131	0.07	0.49	0.49	0.9
UPRR Triple Box to Ames Ave	19158	0.06	1.08	1.08	1.08
Ames Ave to Calaveras Blvd	16924	0.06	0.19	0.19	0.9

As seen in Table 5-11, the total stage-discharge uncertainties range from 0.36 to 1.08 feet. The minimum uncertainty in stage based on a fair Manning's n-value reliability and cross sections based on topographic mapping is 0.9 ft per Table 5-2 from EM 1110-2-1619. All but two of the calculated total stage-discharge uncertainties listed in Table 3.8 are lower than the minimum value and were deemed too low. A total stage-discharge uncertainty of 0.9 was adopted for each index location with a total stage-discharge uncertainty value below 0.9 ft. This increase in the stage-discharge uncertainty was to account for effects that are not explicitly accounted for in the calculations described in this chapter. These effects include:

changes in n-value during the event, unanticipated debris inflow, sediment transport and volume during events among a few. For the two index locations where the total stage-discharge uncertainty was above 0.9 ft the computed total stage-discharge uncertainty was deemed acceptable and used.

5.3.3 Discharge-Probability Uncertainty

The uncertainty of the discharge-probability relationship was developed using the HEC-FDA graphical approach. HEC-FDA computes the uncertainty in terms of confidences limits based on an equivalent period of record for ungauged watersheds. The equivalent period of record used for Berryessa Creek was 35 years for all index sections.

5.3.4 Economic Inputs

HEC-FDA is primarily used as a flood damage analysis tool, of which project performance is one aspect; therefore, economic inputs in the form of stage-damage curves and floodplain structure locations are required. The economic inputs are independent of the project performance results. To use the model for project performance purposes HEC-FDA requires a minimum of one hypothetical damage curve and one hypothetical structure to be entered into the model. The economic data entered into the model consisted of one data point and was used only to allow the calculation of the CNP; it did not affect the performance evaluation or represent any particular structure in the floodplain.

5.3.5 Target Stages

The top of levee ground elevations were used as the target stages for the HEC-FDA program to determine the CNP for each reach of each alternative. The higher of the bank elevation or the top of levee/floodwall elevation was used as the target stage for each section.

5.3.6 Results

The HEC-FDA model was run for Alternatives 2A/d, 2B/d, and 4B/d to determine if the project performance met the requirements for the study. Alternative 2A/d was required to achieve a minimum CNP of 50% for the 1% chance exceedance event for each of the five reaches, and Alternatives 2B/d and 4/d were required to achieve 90%. Alternative 2A/d was run using hydrologic inputs assuming no future improvements upstream of I-680. Alternatives 2B/d and 4/d were run using hydrologic inputs assuming a locally constructed bypass structure is in-place upstream of I-680

If the minimum CNP was not achieved, the alternative was refined as necessary, the inputs recomputed, and the HEC-FDA model rerun. This process was repeated until the minimum CNP requirement was met. Table 3.9 lists the 1% chance exceedance water surface elevation (WSEL), target stage elevation, and resulting CNP for the 1% chance exceedance event for Alternatives 2A/d, 2B/d, and 4/d. As seen in the Table 5-12, all reaches meet or exceed the minimum CNP of 50% for the 1% chance exceedance event required for Alternative 2A/d and 90% for Alternatives 2B/d and 4/d.

Table 5-12 Conditional Non-Exceedance Probability Results

	Reach				
	I-680 to Montague Blvd	**Montague to UPRR Trestle**	**UPRR Trestle to UPRR Triple Box**	**UPRR Triple Box to Ames Ave**	**Ames Ave to Calaveras Blvd**
Index Section	**22274**	**21601**	**20131**	**19158**	**16924**
Alternative 2A/d					
1% Chance Exceedance WSEL	64.15	60.06	53.56	50.74	38.20
Target Stage (ft)	65.27	62.43	57.11	52.55	38.88
Computed CNP for 1% event	76%	82%	99%	97%	79%
Alternative 2B/d					
1% Chance Exceedance WSEL	63.38	61.41	53.84	48.83	39.35
Target Stage (ft)	65.45	63.90	57.15	52.55	41.35
Computed CNP for 1% event	95%	99%	100%	95%	95%
Alternative 4/d					
1% Chance Exceedance WSEL	63.02	61.70	53.50	49.91	41.30
Target Stage (ft)	66.00	64.70	57.11	52.55	43.80
Computed CNP for 1% event	97%	96%	99%	96%	97%

5.4 Results

This section summarizes the hydraulic characteristics for the without-project conditions and alternatives. Further details on cross sections, quantities and costs are included in *Appendix B, Part IV: Design and Cost of Alternatives*. All project features were modeled individually to determine the associated hydraulic effects prior to combining the features into composite with-project alternative models. Summary results of the hydraulic parameters, averaged by reach[9], are presented in Table 5-13. Additional details can be found in Table 5-14 and in the accompanying HEC-RAS model.

[9] The hydraulic reaches discussed in this appendix refer to the hydraulic reaches specified in the scope of work to ensure hydraulic performance goals were met. The Economic Appendix discusses the results of the economic analysis on economic reaches developed independently of the hydraulic reaches, based on economic criteria. The reaches referenced in this and the economic appendix are independent and are not meant to correlate between appendices.

Table 5-13 Summary of Reach Average Hydraulic Results for the 1% Chance Exceedance Event

Reach from	Reach to	Without-Project		Alternative 2A/d		Alternative 5	
		Vel	Depth	Vel	Depth	Vel	Depth
		(ft/s)	(ft)	(ft/s)	(ft)	(ft/s)	(ft)
US Extent	Old Piedmont Road	8.0	6.8	Same as Without-Project Condition		8.2	6.6
Old Piedmont Road	Piedmont-Cropley	10.9	7.4			15.7	7.6
Piedmont-Cropley	Drop Structure US of Morrill Ave.	6.6	6.2			6.6	6.9
Drop Structure US of Morrill Ave.	Morrill Ave.	4.9	12.1			11.3	9.6
Morrill Ave.	Cropley Ave.	5.1	9.8			8.2	9.8
Cropley Ave.	I-680	12.2	6.9			12.9	7.7
I-680	Montague Expy	6.2	4.8	6.9	5.0	10.9	5.0
Montague Expy	UPRR Trestle	6.3	5.7	6.6	5.0	9.6	4.7
UPRR Trestle	UPRR Culvert	8.2	5.3	8.1	5.5	13.7	5.0
UPRR Culvert	Ames Avenue	5.9	5.0	6.5	5.2	7.6	5.5
Ames Avenue	Yosemite Drive	7.4	4.7	9.4	4.8	14.6	4.5
Yosemite Drive	Los Coches Street	5.7	4.1	7.2	4.7	13.3	5.6
Los Coches St	Calaveras Blvd	5.4	6.5	5.2	6.9	6.4	8.6
Calaveras Blvd	Downstream Extent	5.3	4.0	5.3	5.0	6.5	5.2

Table 5-13 Summary of Reach Average Hydraulic Results for the 1% Chance Exceedance Event (cont.)

Reach from	Reach to	Alternative 2B/d without Future Upstream of I-680 Improvements		Alternative 2B/d with Future Upstream of I-680 Improvements	
		Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)
US Extent	I-680	Same as Without-Project Condition		Hydraulic Analysis not Conducted for Locally Developed Future Upstream Improvements[1]	
I-680	Montague Expy	8.1	4.0	8.9	4.8
Montague Expy	UPRR Trestle	6.2	4.8	6.6	5.7
UPRR Trestle	UPRR Culvert	9.0	4.7	10.0	5.5
UPRR Culvert	Ames Avenue	7.1	3.7	8.0	4.4
Ames Avenue	Yosemite Drive	8.0	3.6	9.0	4.3
Yosemite Drive	Los Coches Street	7.7	4.2	8.5	5.1
Los Coches St	Calaveras Blvd	9.4	4.4	9.3	5.3
Calaveras Blvd	Downstream Extent	5.2	5.2	4.8	6.2

Note: 1. Locally Developed Future Upstream Improvements are described in Section 5.2.2.

Table 5-13 Summary of Reach Average Hydraulic Results for the 1% Chance Exceedance Event (cont.)

Reach from	Reach to	Alternative 4/d without Future Upstream of I-680 Improvements		Alternative 4/d with Future Upstream of I-680 Improvements	
		Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)
US Extent	I-680	Same as Without-Project Condition		Hydraulic Analysis not Conducted for Locally Developed Future Upstream Improvements[1]	
I-680	Montague Expy	6.5	4.2	7.3	5.3
Montague Expy	UPRR Trestle	4.8	5.4	5.0	7.4
UPRR Trestle	UPRR Culvert	6.6	5.2	7.4	6.6
UPRR Culvert	Ames Avenue	5.3	4.4	6.2	5.5
Ames Avenue	Yosemite Drive	6.3	4.4	7.1	5.5
Yosemite Drive	Los Coches Street	6.4	5.9	6.9	7.6
Los Coches St	Calaveras Blvd	8.7	5.8	9.3	6.6
Calaveras Blvd	Downstream Extent	5.8	5.0	5.5	6.0

Note: 1. Locally Developed Future Upstream Improvements are described in Section 5.2.2.

These results are for fully contained flows. Comparison to the without-project conditions is therefore hypothetical only; the computed without-project water surface elevation at any point assumes full containment at each upstream section, and flows are restricted to the extent of each cross section in the event of breakout. Results accounting for breakout flows are presented in *Appendix B, Part II: Floodplain Development*, and *Appendix B, Part III: Geomorphology*.

Table 5-14 With-Project Hydraulic Results Summary for the 1% Chance Exceedance Event

FLO-2D Grid Cell / HEC-RAS Station	Without-Project		Alternative 2A/d		Alternative 5	
	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)
US Extent						
3142[1]	5.57	8.31	Same as Without-Project Condition		5.6	8.15
3141[1]	6.59	6.94			6.85	7.36
3140[1]	8.12	5.88			11.06	5.78
3111[1]	8.99	6.09			11.01	4.9
3110[1]	8.99	4.67			8.57	2.67
3109[1]	7.2	6.18			8.91	3.56
3108[1]	8.82	4.89			8.12	7.33
3107[1]	8.82	6.03			6.47	9.01
3106[1]	8.72	12.25			7.04	10.31
Old Piedmont Road						
3075[1]	11.99	4.18	Same as Without-Project Condition		11.1	7.49
3104[1]	11.99	6.25			13.74	7.65
3103[1]	13.72	7.71			17.4	6.81
3072[1]	13.72	5.41			17.4	5.37
3071[1]	8.42	8.36			15.2	8.09
3039[1]	8.29	8.35			17.58	5.64
3038[1]	8.42	11.6			17.58	12.39
Piedmont-Cropley						
2967[1]	5.65	5.9	Same as Without-Project Condition		3.78	5.22
2966[1]	6.24	4.82			3.85	5.63
2930[1]	6.24	5.21			5.43	6.17
2893[1]	5.08	5.55			5.27	6.2
2892[1]	4.59	6.01			4.71	6.52
2891[1]	4.94	5.8			4.45	6.51
2853[1]	6.35	6.1			6.22	6.75
2852[1]	7.98	5.24			7.58	5.61
2851[1]	7.98	5.54			7.87	5.93
2850[1]	6.8	7.58			6.37	8.05
2811[1]	6.8	6.9			6.37	7.45
2771[1]	6.02	7.88			5.68	8.37
2729[1]	6.75	7.46			6.54	7.82
2728[1]	6.93	5.49			6.72	5.74
2727[1]	6.46	6			6.71	6.49
2685[1]	7.51	4.95			7.61	5.42
2642[1]	7.55	6.35			7.63	6.73
2599[1]	7.13	6.25			7.08	6.78
2555[1]	7.17	6.57			7.3	7.32
2512[1]	7.42	7.18			7.27	7.96

FLO-2D Grid Cell / HEC-RAS Station	Without-Project		Alternative 2A/d		Alternative 5	
	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)
2468[1]	7.97	6.53			8.21	7.47
2467[1]	8.36	6.44			9.32	7.03
2423[1]	7.55	6.26			7.45	6.62
2379[1]	7.54	6.29			7.49	6.88
2334[1]	5.5	7.45			5.49	8.26
2291[1]	6.47	6.36			6.47	7.21
2247[1]	7.53	6.73			7.43	7.41
2202[1]	7.52	4.98			7.43	5.73
2158[1]	6.76	5.05			7.22	6.29
2114[1]	6.18	6.52			6.87	7.47
2069[1]	5.02	7.69			5.61	8.35
2024[1]	5.53	7.23			5.77	7.77
1979[1]	6.01	6.17			6.29	6.67
1934[1]	6.04	5.63			6.5	6.19
1888[1]	5.79	5.23			6.5	5.75
1842[1]	5.95	7.39			6.46	7.95
1797[1]	5.95	6.86			6.46	7.51
1751[1]	5.38	6.86			5.97	7.42
1705[1]	5.24	6.08			5.81	6.7
1659[1]	4.67	6.7			5.52	7.19
1613[1]	9.12	6.52			9.71	7
1566[1]	9.12	4.52			9.78	6.45
Drop Structure US of Morrill Ave.						
1471[1]	3.83	11.19	Same as Without-Project Condition		13.18	9.59
1424[1]	5.48	11.5			14.14	6.74
1375[1]	5.48	11.01			14.14	9.38
1326[1]	4.92	13.32			8.72	11.35
1279[1]	4.92	13.36			6.29	10.7
Morrill Ave.						
1230[1]	5.9	8.67	Same as Without-Project Condition		9.94	7.26
1182[1]	5.9	8.23			9.94	9.46
1134[1]	4.55	9.1			7.93	9.99
1085[1]	5.25	9.35			7.51	10.14
1035[1]	5.42	9.57			7.45	10.24
986[1]	4.82	10.24			7.47	10.33
938[1]	4.8	10.09			7.64	10.43
889[1]	4.99	11.17			7.77	10.33
890[1]	4.19	11.9			7.71	10.24
Cropley Ave.						
840[1]	8.34	8.8	Same as Without-		8.04	10.47

FLO-2D Grid Cell / HEC-RAS Station	Without-Project		Alternative 2A/d		Alternative 5	
	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)
788[1]	10.48	7.05	Project Condition		9.22	8.76
736[1]	11.59	6.64			10.01	8.41
683[1]	11.59	6.46			10.25	8.22
630[1]	12.46	7			11.54	8.35
577[1]	12.46	6.46			12.71	7.95
524[1]	11.97	7.22			13.25	7.5
471[1]	10.17	7.57			13.26	7.27
418[1]	10.85	7.26			13.04	7.37
364[1]	13.1	6.6			13.46	7.52
309[1]	13.53	5.96			13.55	7.09
257[1]	13.53	5.84			13.55	6.94
211[1]	10.83	8			12.87	7.91
168[1]	11.85	7.31			12.87	7.82
130[1]	11.86	7.08			12.52	8.08
96[1]	16.51	8.15			18	8.89
68[1]	16.52	2.11			18	1.36
43[1]	12.37	7.86			16.83	9.05
I-680						
25296[2]	5.2	4.7	5.2	4.7	12.9	3.6
25245[2]	5.8	5.1	5.8	5.1	14.5	3.5
25155[2]	6.2	5.0	6.2	5.0	14.5	4.0
24997[2]	7.1	4.9	7.1	5.0	15.3	3.9
24886[2]	6.9	5.6	6.8	5.6	16.1	3.8
24791[2]	7.1	5.0	7.0	4.9	17.1	3.6
24694[2]	7.9	4.6	7.8	4.6	19.1	3.4
24171[2]	7.8	5.1	8.5	4.8	15.4	3.8
24079[2]	7.5	4.8	9.6	4.4	15.3	3.8
23986[2]	6.9	5.0	9.1	4.4	15.2	3.8
23889[2]	4.3	4.6	9.1	4.4	15.1	3.8
23786[2]	4.9	4.1	9.1	4.4	14.8	3.9
23710[2]	5.7	4.2	9.1	4.4	14.6	3.9
23610[2]	6.9	4.6	9.1	4.4	14.2	4.0
23522[2]	5.4	5.2	9.1	4.4	13.8	4.0
23413[2]	5.9	4.4	9.1	4.4	13.2	4.2
23326[2]	7.3	4.2	9.1	4.4	12.7	4.3
23185[2]	5.6	3.5	9.1	4.4	11.7	4.5
23062[2]	5.6	4.9	9.1	4.4	4.9	6.2
22951[2]	7.8	4.3	7.4	4.4	3.9	7.1
22865[2]	9.2	4.4	6.9	4.6	3.8	7.2
22806[2]	8.3	4.6	6.5	4.7	3.8	7.2

FLO-2D Grid Cell / HEC-RAS Station	Without-Project		Alternative 2A/d		Alternative 5	
	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)
22748[2]	8.5	4.8	6.2	4.8	3.8	7.3
22693[2]	8.1	4.1	5.8	4.9	3.8	7.3
22603[2]	8.8	4.8	5.3	5.2	3.7	7.3
22274[2]	3.0	5.4	3.9	6.0	3.5	7.5
22117[2]	4.4	5.3	3.3	6.5	3.5	7.6
21883[2]	2.4	5.9	2.7	7.4	3.4	7.7
21873[2]	2.2	6.2	2.7	7.5	3.4	7.7
21864[2]	1.9	6.2	2.6	7.5	3.4	7.7
21852[2]	1.7	3.4	2.6	7.6	3.4	7.7
21844[2]	1.8	3.5	2.6	7.6	3.3	7.7
21832[2]	1.9	4.6	2.6	7.7	3.3	7.7
21821[2]	5.6	11.4	6.6	8.9	4.7	8.9
Montague Expressway						
21667[2]	6.3	11.0	9.7	6.5	7.8	6.3
21657[2]	4.1	6.6	8.1	5.4	12.5	5.3
21646[2]	3.8	5.8	8.0	5.4	12.5	5.3
21634[2]	4.3	4.9	8.0	5.5	12.5	5.3
21623[2]	6.6	5.5	7.9	5.5	12.5	5.3
21601[2]	8.9	5.6	7.9	5.5	12.4	5.3
21314[2]	7.5	6.2	6.9	5.9	11.8	5.4
21276[2]	5.6	6.9	6.8	5.9	11.7	5.5
21270[2]	5.1	6.7	5.6	8.2	6.4	8.4
UPRR Trestle						
21226[2]	6.5	6.5	6.5	8.0	7.9	7.0
21219[2]	7.4	6.1	9.1	5.1	15.4	4.7
21203[2]	8.4	5.7	9.1	5.1	15.4	4.7
21050[2]	10.3	4.8	9.1	5.1	15.4	4.7
20823[2]	10.1	5.3	9.2	5.1	15.4	4.7
20595[2]	7.6	5.5	9.2	5.1	15.3	4.7
20368[2]	8.7	5.0	8.7	5.2	15.1	4.8
20131[2]	5.9	5.8	8.1	5.4	14.9	4.8
19901[2]	8.2	5.1	7.9	5.5	14.6	4.9
19676[2]	10.1	4.9	7.1	5.8	14.1	4.9
19413[2]	5.7	5.2	6.2	6.3	5.0	7.3
19400[2]	6.3	5.1	6.1	6.3	4.8	7.4
19390[2]	4.0	10.1	5.3	10.4	4.6	7.7
UPRR Triple Box						
19285[2]	4.4	5.3	7.3	5.7	6.0	6.5
19268[2]	3.7	5.8	7.3	5.7	5.9	6.5
19244[2]	3.6	6.1	7.3	5.7	5.8	6.6

FLO-2D Grid Cell / HEC-RAS Station	Without-Project		Alternative 2A/d		Alternative 5	
	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)
19234[2]	3.6	6.0	7.3	5.7	5.8	6.6
19184[2]	6.3	5.4	7.3	5.7	5.7	6.7
19172[2]	6.8	5.6	7.3	5.7	5.6	6.7
19158[2]	7.0	5.9	7.3	5.7	5.6	6.8
19083[2]	8.6	5.6	7.2	5.7	11.1	5.5
18904[2]	6.5	5.2	7.3	5.7	9.6	5.9
18881[2]	5.7	5.1	7.4	6.2	9.4	6.0
Ames Ave						
18805[2]	8.8	5.7	10.6	5.1	18.7	4.1
18774[2]	11.7	4.8	9.7	5.0	17.2	4.3
18553[2]	9.2	4.7	9.6	5.0	16.8	4.3
18259[2]	7.5	4.7	9.6	4.9	15.8	4.5
18045[2]	7.7	4.4	9.6	4.9	14.6	4.7
17811[2]	4.1	4.5	9.9	4.8	13.0	5.0
17602[2]	6.5	6.0	10.4	4.7	11.3	5.4
17571[2]	6.2	6.3	6.3	5.5	9.4	5.8
Yosemite Dr						
17470[2]	5.7	5.4	7.2	5.1	14.6	4.5
17448[2]	6.9	5.1	7.0	4.1	14.5	4.5
17427[2]	10.6	4.9	8.1	3.9	15.6	4.5
17281[2]	8.5	3.3	8.1	3.9	15.5	4.5
16924[2]	7.1	4.6	8.2	3.9	15.1	4.6
16654[2]	5.8	4.7	5.6	4.8	14.5	4.7
16437[2]	4.7	4.4	8.5	4.3	16.3	5.2
16139[2]	9.5	4.3	8.5	4.3	15.8	5.3
15928[2]	5.3	3.7	8.5	4.3	15.3	5.4
15665[2]	3.6	4.4	8.3	4.4	14.6	5.5
15398[2]	4.4	3.3	7.6	4.7	13.6	5.8
15156[2]	3.4	4.3	6.8	5.2	12.5	6.0
14944[2]	5.4	3.5	6.1	5.8	11.5	6.3
14685[2]	4.7	4.3	5.3	6.6	8.2	7.8
14467[2]	4.3	4.3	4.7	7.4	5.0	10.5
14422[2]	2.5	7.2	4.4	7.9	5.9	10.2
Los Coches St.						
14350[2]	2.8	7.3	6.1	7.3	6.9	8.8
14179[2]	6.0	5.9	6.2	6.5	6.8	8.9
14121[2]	7.0	6.5	5.3	7.3	6.6	9.1
13937[2]	6.3	8.2	4.8	8.3	7.0	9.1
13887[2]	5.7	9.0	4.0	9.4	7.3	12.0
Calaveras Blvd						

FLO-2D Grid Cell / HEC-RAS Station	Without-Project		Alternative 2A/d		Alternative 5	
	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)
13741[2]	12.4	4.4	8.0	5.7	15.1	6.0
13653[2]	5.1	5.4	6.9	6.4	6.3	6.9
13603[2]	5.0	5.5	6.2	6.9	6.2	7.0
13553[2]	5.0	5.5	6.1	6.9	6.1	7.0
13503[2]	4.7	5.8	5.8	7.2	5.9	7.3

Note: 1. FLO-2D Grid Cell

2. HEC-RAS Cross Section Station

Table 5-14 With-Project Hydraulic Results Summary for the 1% Chance Exceedance Event (cont.)

Station	Alternative 2B/d without Future Upstream of I-680 Improvements		Alternative 2B/d 5 with Future Upstream of I-680 Improvements		Alternative 4/d 5 without Future Upstream of I-680 Improvements		Alternative 4/d 5 with Future Upstream of I-680 Improvements	
	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)
US Extent to I-680								
	Same as Without-Project Condition listed in Table 5-13		Hydraulic Analysis not Conducted for Locally Developed Future Upstream Improvements[1]		Same as Without-Project Condition listed in Table 5-13		Hydraulic Analysis not Conducted for Locally Developed Future Upstream Improvements[1]	
I-680								
25296	3.8	5.4	4.3	6.5	6.3	4.5	6.6	4.8
25245	9.3	5.0	10.3	5.7	8.2	5.0	9.3	6.1
25155	9.4	5.0	10.5	5.6	8.2	5.0	9.4	6.1
24997	9.9	4.9	11.0	5.5	8.3	4.9	9.5	6.0
24886	10.5	4.7	11.6	5.3	8.4	4.8	9.7	5.9
24791	11.0	4.6	12.2	5.2	8.5	4.8	9.9	5.8
24694	12.2	4.4	13.4	5.0	8.8	4.6	10.3	5.5
24171	9.5	4.0	10.6	4.6	6.9	3.9	8.0	4.7
24079	7.9	4.3	9.3	4.8	7.2	3.7	8.4	4.5
23986	9.3	3.4	10.5	3.9	6.5	3.7	7.4	4.5
23889	9.6	3.3	10.8	3.9	6.5	3.7	7.4	4.5
23786	10.3	3.1	11.5	3.7	6.5	3.7	7.4	4.5
23710	11.3	2.9	12.4	3.5	6.5	3.7	7.4	4.5
23610	9.1	3.1	10.0	3.7	6.5	3.7	7.4	4.5
23522	8.0	3.2	8.9	3.8	6.5	3.7	7.4	4.5
23413	7.8	3.3	8.6	3.9	6.5	3.7	7.4	4.5
23326	7.4	3.4	8.2	4.1	6.5	3.7	7.4	4.5
23185	7.9	3.2	8.6	3.9	6.5	3.7	7.4	4.5
23062	7.6	3.3	7.8	4.1	6.5	3.7	7.2	4.5
22951	7.0	3.5	7.1	4.4	6.5	3.7	7.0	4.6
22865	6.4	3.7	6.6	4.6	6.4	3.7	6.9	4.7
22806	6.0	3.9	6.2	4.8	6.4	3.7	6.7	4.8
22748	5.6	4.1	5.9	5.0	6.4	3.7	6.6	4.9
22693	6.3	4.0	6.5	4.9	6.4	3.7	6.4	5.1
22603	7.2	4.0	7.4	4.9	6.5	3.7	6.3	5.3
22274	7.3	4.1	6.9	5.2	4.6	4.4	5.2	6.4
22117	6.3	4.1	7.2	5.2	4.0	5.1	4.7	7.1
21883	5.0	4.7	6.0	5.8	3.3	6.1	4.1	8.2
21873	4.9	4.8	5.9	5.9	3.2	6.2	4.0	8.3
21864	4.9	4.8	5.9	5.9	3.2	6.2	4.0	8.3

Station	Alternative 2B/d without Future Upstream of I-680 Improvements		Alternative 2B/d 5 with Future Upstream of I-680 Improvements		Alternative 4/d 5 without Future Upstream of I-680 Improvements		Alternative 4/d 5 with Future Upstream of I-680 Improvements	
	Vel	Depth	Vel	Depth	Vel	Depth	Vel	Depth
	(ft/s)	(ft)	(ft/s)	(ft)	(ft/s)	(ft)	(ft/s)	(ft)
21852	4.8	4.8	5.8	5.9	3.2	6.3	4.0	8.4
21844	4.8	4.9	5.8	6.0	3.2	6.3	4.0	8.4
21832	4.7	4.9	5.7	6.0	3.1	6.4	3.9	8.5
21821	3.7	6.9	4.2	8.2	5.9	5.8	6.1	8.0
Montague Expressway								
21667	5.2	6.3	5.7	7.4	6.2	5.8	6.4	8.0
21657	8.3	5.3	8.8	6.2	6.1	5.8	6.3	8.1
21646	8.3	5.3	8.8	6.2	6.0	5.8	6.2	8.1
21634	8.3	5.3	8.7	6.3	6.0	5.9	6.2	8.1
21623	8.2	5.3	8.7	6.3	6.0	5.9	6.2	8.2
21601	8.2	5.3	8.7	6.3	5.9	5.9	6.1	8.2
21314	7.3	5.7	7.7	6.7	5.2	6.8	5.5	9.2
21276	7.2	5.7	7.6	6.7	5.1	6.9	5.4	9.3
21270	4.3	7.9	4.9	9.0	3.5	8.4	3.8	10.9
UPRR Trestle								
21226	4.5	7.6	5.3	8.6	4.4	6.7	5.2	8.0
21219	9.2	5.1	10.1	5.9	7.2	4.9	8.3	6.1
21203	9.2	5.1	10.1	5.9	7.2	4.9	8.3	6.1
21050	9.2	5.1	10.1	5.9	7.2	4.9	8.2	6.1
20823	9.2	5.1	10.2	5.9	7.2	4.9	8.2	6.1
20595	9.4	5.1	10.5	5.8	7.2	4.9	8.2	6.2
20368	9.9	4.9	11.1	5.6	7.2	4.9	8.1	6.2
20131	11.0	4.7	12.3	5.3	7.1	5.0	7.9	6.4
19901	8.6	4.0	9.5	4.8	7.0	5.0	7.6	6.6
19676	8.5	4.0	9.2	4.9	5.4	5.5	5.8	7.2
19413	8.0	4.2	8.6	5.2	4.7	6.2	5.2	8.0
19400	8.0	4.2	8.5	5.2	4.7	6.3	5.2	8.1
19390	4.7	7.3	5.7	8.6	6.2	8.2	7.4	9.9
UPRR Triple Box								
19285	7.4	4.3	8.4	5.1	5.7	5.1	6.7	6.3
19268	7.5	4.3	8.5	5.1	5.8	5.1	6.7	6.2
19244	7.5	4.3	8.5	5.0	5.8	5.1	6.7	6.2
19234	7.5	4.3	8.5	5.0	5.8	5.1	6.7	6.2
19184	7.6	4.2	8.6	5.0	5.8	5.0	6.7	6.2
19172	7.6	4.2	8.7	5.0	5.8	5.0	6.8	6.2
19158	7.7	4.2	8.7	5.0	5.8	5.0	6.8	6.2
19083	7.9	4.1	9.0	4.9	5.9	5.0	6.8	6.1

Station	Alternative 2B/d without Future Upstream of I-680 Improvements		Alternative 2B/d 5 with Future Upstream of I-680 Improvements		Alternative 4/d 5 without Future Upstream of I-680 Improvements		Alternative 4/d 5 with Future Upstream of I-680 Improvements	
	Vel	Depth	Vel	Depth	Vel	Depth	Vel	Depth
	(ft/s)	(ft)	(ft/s)	(ft)	(ft/s)	(ft)	(ft/s)	(ft)
18904	8.9	3.7	10.0	4.5	6.1	4.8	7.1	5.9
18881	4.9	4.8	5.8	5.6	6.1	4.7	7.1	5.9
Ames Ave								
18805	4.6	4.8	5.5	5.5	6.9	4.2	7.9	5.3
18774	8.6	3.6	9.6	4.3	6.8	4.3	7.8	5.3
18553	8.6	3.6	9.6	4.3	6.8	4.3	7.8	5.4
18259	8.5	3.6	9.6	4.3	6.7	4.4	7.6	5.5
18045	8.5	3.6	9.6	4.3	6.5	4.5	7.3	5.7
17811	8.5	3.6	9.5	4.4	6.1	4.8	6.8	6.1
17602	8.2	3.7	9.1	4.5	5.5	5.4	6.2	6.7
17571	4.9	5.7	5.8	6.7	5.3	5.5	6.1	6.8
Yosemite Dr								
17470	4.6	5.7	5.8	6.8	5.6	5.1	6.4	6.5
17448	6.2	4.0	7.3	4.7	5.6	5.2	6.4	6.5
17427	7.5	3.7	8.2	4.5	6.2	5.0	6.9	6.4
17281	7.3	3.8	8.0	4.6	6.0	5.2	6.7	6.6
16924	6.3	4.3	6.9	5.3	5.4	5.8	6.1	7.2
16654	5.3	4.9	6.0	6.0	4.9	6.4	5.6	7.9
16437	8.6	4.3	9.6	5.2	7.4	5.8	8.4	7.1
16139	8.6	4.3	9.6	5.2	7.4	5.9	8.3	7.2
15928	8.6	4.3	9.6	5.2	7.4	5.9	8.1	7.3
15665	8.6	4.3	9.6	5.2	7.3	5.9	7.9	7.5
15398	8.6	4.3	9.6	5.2	7.2	6.1	7.6	7.8
15156	8.5	4.3	9.6	5.2	6.9	6.2	7.2	8.3
14944	8.6	4.3	9.7	5.1	6.7	6.5	6.8	8.7
14685	8.6	4.3	9.7	5.0	6.2	7.0	6.3	9.4
14467	8.5	4.3	7.3	5.0	5.7	7.5	5.9	10.0
14422	7.4	5.4	8.0	6.2	8.4	6.6	7.6	9.5
Los Coches St.								
14350	7.7	5.2	8.3	6.0	10.3	5.4	11.4	6.3
14179	11.1	4.3	11.4	5.1	9.4	5.9	10.0	6.8
14121	10.6	4.5	10.7	5.4	8.8	6.4	9.7	7.0
13937	10.5	4.7	8.9	6.2	8.8	6.7	8.7	7.8
13887	7.5	6.2	6.6	7.8	8.5	6.9	8.2	8.1
Calaveras Blvd								
13741	8.0	5.8	7.5	7.1	11.4	5.2	11.6	6.2
13653	6.3	6.9	5.9	8.3	6.3	6.9	5.8	8.3

Station	Alternative 2B/d without Future Upstream of I-680 Improvements		Alternative 2B/d 5 with Future Upstream of I-680 Improvements		Alternative 4/d 5 without Future Upstream of I-680 Improvements		Alternative 4/d 5 with Future Upstream of I-680 Improvements	
	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)	Vel (ft/s)	Depth (ft)
13603	6.2	7.0	5.8	8.4	6.2	7.0	5.7	8.4
13553	6.1	7.0	5.7	8.5	6.1	7.0	5.6	8.4
13503	5.9	7.3	5.5	8.8	5.8	7.3	5.4	8.7

Note: 1. Locally Developed Future Upstream Improvements are described in Section 5.2.2.

CHAPTER 6: SUMMARY

This appendix (*Appendix B, Part I: Hydraulic Analysis of Alternatives*) presents the modeling input and results for the without-project and project alternatives hydraulics. Modeling in this portion of the engineering appendix is based both on steady-state, 1-dimensional flow for the original modeling (retained for continuity) and unsteady, 1-dimensional flow for the revised GRR modeling. The total stage-discharge uncertainties for six index reaches upstream of I-680 and five index reaches downstream of I-680 were developed for the without-project and project alternatives for use in Economic analysis as presented in *Appendix C: Economics.* In addition, the stage-discharge uncertainties were used to size project alternatives 2A/d, 2B/d, and 4/d using risk-based principles.

The hydraulic results documented in this appendix were applied to the development of the floodplain mapping for the without-project and project alternatives. Details on the 2-dimensional modeling and mapping of overflows are presented in *Appendix B, Part II: Floodplain Development.*

Readers are referred to HDR, Inc.'s Technical Memorandum, *Berryessa Creek Hydraulic Analysis,* (HDR 2004) for details on the initial development and calibration of the without-project HEC-RAS steady-state model. Tetra Tech Inc.'s Technical Memorandum to U.S. Army Corps of Engineers, *Changes to without-project Hydraulic Modeling* (Tetra Tech 2005) details the changes made by Tetra Tech to the HDR HEC-RAS model which serve as the basis for the modeling reported in this document.

The following refinements for the selected plan during the detailed design phase are routinely carried out:

- Obtain updated topographic data to ensure that all channel breaklines are properly identified.
- Conduct detailed survey of bridge and culvert crossings.
- Model calibration is recommended if high water events occur and high water marks can be measured during the peak flow event.

CHAPTER 7: REFERENCES

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